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Water-Softening and Filtration Plant of the Metropolitan Water District of Southern California

By James M. Montgomery and William W. Aultman

THERE are few people, even in Southern California, who realize that the region adjacent to Los Angeles is naturally a desert. The luxuriant subtropical vegetation of the cultivated areas, particularly when contrasted with the surrounding deserts, creates an illusion of abundant moisture. It takes much more than a casual inspection to reveal the labor that lies behind this illusion.

As early as 1904 it was evident that if Los Angeles continued to grow as it had during the preceding three decades a supplemental water supply was an absolute necessity. Investigations showed that the Owens River was the best available source of supply. In 1905 a bond issue of $1,500,000 was passed for preliminary investigations; in June, 1907, a $23,000,000 bond issue was voted by a majority of ten to one to cover the construction of a 233-mile aqueduct with a designed capacity of 400 cubic feet per second; and in 1913 the Owens River Aqueduct was completed.

The new supply was expected to solve the city's water problem for many years to come, but agriculture, population, and industries, all growing, demanded more water. With present sources being overdrawn continually, it was clear that a new importation to meet the needs not only of Los Angeles but of the entire Southern California metropolitan area must be made. The only new source available was the Colorado River. Accordingly, upon his request, William Mulholland, Chief Engineer of the Bureau of Waterworks

A paper presented on October 26, 1939, at the California Section Meeting at San Francisco, by James M. Montgomery, Consulting Engineer, Hoover and Montgomery, Columbus, Ohio; and by William W. Aultman, Engineer, Metropolitan Water District of Southern California.

and Supply of the City of Los Angeles, was authorized to make preliminary surveys of an aqueduct route from the Colorado River; and in June, 1925, the voters of Los Angeles authorized a bond issue of $2,000,000 to continue this investigation. During the period from 1923 until March, 1929, all work for the project was under the direction of William Mulholland and H. A. Van Norman of the Los Angeles Bureau of Waterworks and Supply.

The smaller communities in Southern California, although having just as serious water shortage, were not in a financial position to make importations such as Los Angeles had accomplished in 1913. The Colorado River Aqueduct was, therefore, planned not as a Los Angeles project but as a Southern California enterprise, not on the basis of meeting immediate needs alone, but on the far broader basis of insuring for generations to come an adequate water supply for the region as a whole.

Such is the background of the Metropolitan Water District of Southern California which was formed under an enabling act passed by the California State Legislature in May, 1927, and which provided for the joining together of municipalities or water districts for the purpose of developing and maintaining a water supply. Permanent organization of the District was accomplished February 9, 1929, and since March, 1929, all engineering work has been under the direction of F. E. Weymouth, General Manager and Chief Engineer.

Boulder Canyon Project

To trace the progress of the Colorado River Aqueduct development it is also necessary to trace the progress of the Boulder Canyon Project, for without the Boulder Dam—its storage of flood flows previously lost into the ocean, and its production of cheap power—the aqueduct as finally constructed would not have been feasible.

During the period of record, the flow of the Colorado River at Grand Canyon has varied from a minimum of 700 cu.ft. per sec. to a maximum of 127,000 cu.ft. per sec. From the records at other gaging stations along the river and from other data, it has been estimated by the Bureau of Reclamation that a maximum flow of 300,000 cu.ft. per sec. occurred for a short time in 1884. The average discharge at the dam site is 21,000 cu.ft. per sec. with an average annual runoff of 15,069,000 acre feet. Prior to the construction of Boulder Dam, it is estimated that an average of approximately 10-000,000 acre-ft. of water was discharged into the ocean. It is this

water which previously had been lost for consumptive use which is the primary source of water for the aqueduct. And it is the spring flood waters of relatively low mineral content, contributing from one-half to two-thirds of the reservoir inflow, which will produce a mixed reservoir water averaging about 290 parts per million total hardness and about 580 p.p.m. total dissolved solids.

Although it is owned and operated by the federal government, the first reservoir on the Colorado River Aqueduct system is, therefore, Lake Mead formed by Boulder Dam, the highest dam in the world—727 ft. from crest to foundation. The reservoir has a capacity of 30,500,000 acre-ft. or 9,938 billion gallons covering an area of 229 sq.mi. or 146,500 acres, 115 miles long and with a maximum depth of 582 ft. This reservoir is 155 miles upstream from Parker Dam, the structure forming Lake Havasu from which the aqueduct diversion is made.

Colorado River Aqueduct

After the construction of Boulder Dam was assured, the people of the thirteen cities forming the Metropolitan Water District of Southern California—Anaheim, Beverly Hills, Burbank, Compton, Fullerton, Glendale, Long Beach, Los Angeles, Pasadena, San Marino, Santa Ana, Santa Monica, and Torrance—on September 29, 1930, by a majority of five to one, voted a $220,000,000 bond issue to construct the Colorado River Aqueduct. This sum was for an aqueduct with certain features such as tunnels, canals, and conduits, built to ultimate capacity of 1,500 cu.ft. per sec; and with other features, such as inverted siphons, pumping plants, and distribution system, built to only half capacity, or 750 cu.ft. per second.

No attempt to describe the aqueduct in detail will be made here. Many articles have appeared in engineering literature describing the design and construction of this project (2). Only a short summary is included herewith.

The adopted aqueduct project consists of diversion dam, pumping plants, main waterway, storage works, and distributing system. The general arrangement of these features can be seen by referring to maps on pages 22, 23 and 30 of the January, 1939, JOURNAL.

Main Aqueduct

The so-called main aqueduct starts at the Colorado River and terminates at the Cajalco Reservoir, some 10 miles south of the city of Riverside.

The Parker diversion dam is located on the Colorado River just downstream from the mouth of the Bill Williams River in a narrow rock outlet from an excellent storage basin. This dam is unique in that it raises the water only 72 feet above the natural river channel, while the foundations extend 233 feet below the natural river channel, providing the deepest dam foundation in the world. The dam raises the water to an elevation of 450 ft. above sea-level and provides 717,000 acre-ft. or 232 billion gallons of storage for regulating and clarifying the water. In addition to the reservoir formed by Parker Dam (Lake Havasu) there are three other reservoirs along the main aqueduct to the west—Gene Wash Reservoir with a storage capacity of 6,300 acre-ft. or 2 billion gallons; Copper Basin Reservoir with a storage capacity of 24,200 acre-ft. or about 8 billion gallons; and Hayfield Reservoir with a storage capacity of 87,500 acre-ft. or about 29 billion gallons. Cajalco Reservoir, the terminus of the main aqueduct, is considered a part of the distribution system.

From the intake pumping plant, with a static lift of 291 ft., there are to the west along the main aqueduct four additional pumping plants—one at Gene with a lift of 303 ft., one at Iron Mountain with a lift of 144 ft., one at Eagle Mountain with a lift of 438 ft., and one at Hayfield with a lift of 441 ft.—making a total lift of 1,617 ft. From Hayfield, the water flows by gravity to the boundary of each member city.

The initial pump installation consists of three 200 cu.ft. per sec. or 90,000 gal. per min. units in each plant. The units are single stage, single suction, volute type centrifugal pumps, without guide vanes or diffusers. Building space is provided for two additional units and foundations and aqueduct connections for the ultimate installation of nine units per plant, including one standby at ultimate capacity. The delivery pipes from the pumps are 10 ft. in diameter and from 700 to 2,200 ft. long. The main pump motors each operate at 6,900 volts and vary in horsepower rating from 4,300 at the Iron Mountain plant to 12,500 at the Eagle Mountain and Hayfield plants.

Power is delivered from Boulder Dam to the pumping plants over transmission lines owned and operated by the District. These lines will carry ultimately 280,000 kilowatts to the pump motors—enough electricity to light a city larger than Los Angeles. The transmission potential is 230,000 volts.

The main aqueduct is composed of four primary conduit types as

follows: tunnels, 92.1 mi.; inverted siphons or pressure pipes, 28.7 mi.; covered conduit in backfilled trench, 54.4 mi.; open concrete lined canal, 62.8 mi. The remaining 3.6 mi. of the total 241.6-mile length is made up of 1.2 mi. of pump delivery line, 1.1 mi. of unlined canal into Cajalco Reservoir, and 1.3 mi. of passage through reservoirs.

Distribution System

The main aqueduct terminates in the Cajalco Reservoir. From this point the flow is to be conveyed to the member cities by gravity through pressure lines and tunnels which connect with distribution mains or reservoirs of existing systems. Retail deliveries will not be made by the District but by the water departments of the various cities. The ultimate delivery system will consist of a "high line," or upper feeder at a relatively high elevation (for location see page 30 of the January, 1939, JOURNAL), and one or more lines at lower elevations. The first development consists of the high line and such cross-connections and portions of other lines as are required to make initial deliveries. The plan provides maximum flexibility for meeting future shifts in population centers.

Principal terminal storage is provided at Cajalco Reservoir where a capacity of 225,000 acre-ft., or 73 billion gallons, can be made available between the low water level at elevation 1260 and the outlet of the Valverde Tunnel at elevation 1405. The Cajalco site lends itself to development by stages. The initial development, now completed, provides a capacity of 107,000 acre-ft. or about 35 billion gallons, and increments are to be added as required by raising the dam, foundations for which already have been provided. In the ultimate development, this reservoir is designed to have a 75-day hold-over capacity when the aqueduct is operating at its full capacity.

The main upper feeder extends from the Cajalco Reservoir north across the Santa Ana River to the foothills and then in a westerly direction to the eastern boundary of Glendale. This pressure line, with a total length of 62.5 mi., consists of 36 mi. of concrete pipe, 10.3 mi. of steel pipe, and 16.2 mi. of tunnel.

A cross-feeder from Eagle Rock Canyon to the Palos Verdes Hills provides for deliveries to Los Angeles, Compton, Torrance, and Long Beach. Delivery to Pasadena is provided directly from the main upper feeder. A short lateral is required for San Marino

and a reduced capacity extension of the upper feeder to Glendale, Burbank, Beverly Hills and Santa Monica.

This initial system includes the Morris and Palos Verdes Reservoirs. The Morris Reservoir, with a gross storage capacity of 39,300 acre-ft., or about 13 billion gallons, is formed by a dam across the San Gabriel River a short distance north of Azusa. This dam was built by the city of Pasadena for the conservation of flood waters. By purchase agreement, this dam and reservoir without any surface water rights was acquired by the District subject to use by Pasadena pending actual delivery of water through the Colorado River Aqueduct.

The Palos Verdes Reservoir at the extreme south end of the Eagle Rock-Palos Verdes cross-feeder provides 1,000 acre-ft., or 325 million gallons of storage for the absorption of surplus flows, to be fed back at times of peak demand and when the upper portions of the line are closed for repairs.

Water Treatment

As early as 1933–34, studies were made of the softening and filtration problems by the District, under the direction of Arthur Taylor, Consulting Engineer and later Director from Beverly Hills. In 1935–36 an independent investigation was made for the District by the late Harrison P. Eddy of the consulting engineering firm of Metcalf & Eddy upon the need for and methods of softening Colorado River water.

During the past decade, the demands for soft water have, in the west as in other parts of the United States, increased tremendously. Consequently, the District Board of Directors on October 21, 1938, authorized the General Manager and Chief Engineer to proceed with the preparation of plans and specifications for a water softening and filtration plant. The firm of Hoover and Montgomery of Columbus, Ohio, was retained by the District to supervise and advise the District engineering staff in the preparation of plans. On March 10, 1939, the first equipment contracts were awarded. The plans and specifications for the plant have been completed and the contract awarded to the Griffith Company of Los Angeles for $1,632,992 to construct the plant and install the equipment furnished by the District. Actual construction, started on November 13, 1939, is scheduled for completion January 15, 1941.

The softening and filtration plant is to be located on the upper

feeder of the distribution system, near the town of La Verne which is about 30 miles east of Los Angeles. This site was chosen because it is west of the area which may wish to use Colorado River water for agricultural purposes, and which need not be softened; and far enough east so that all the present member cities and most of the potential members may be supplied with softened and filtered water.

The plant has been designed for an initial capacity of 100 million gallons per day, or 155 cu.ft. per sec., and is so constructed, with many of its features of ultimate requirements, that it may be enlarged to 400 million gallons per day, or 620 cu.ft. per sec., its ultimate capacity. When a feeder at a lower elevation is built from Cajalco Reservoir, it will then be necessary to construct another plant on that feeder with an ultimate capacity approximately the same as the La Verne plant.

The Water

Detailed water analyses have been made of the Colorado River water by the U. S. Geological Survey since 1925. Using these available analyses as a basis, the discharge, total hardness relationship was obtained from which the probable hardness of the water since 1902 was estimated. In conjunction with studies on the operation of Boulder Dam, the U. S. Bureau of Reclamation has prepared reservoir operation data indicating how the outflow from Lake Mead would have been controlled, had the dam been built, for the entire period of record. These data, together with the discharge, total hardness relationship, were the basis of a reservoir operation study including full allowance for evaporation and reservoir spill, which indicates that the average total hardness of the water in the reservoir might vary from a minimum of 240 p.p.m. to a maximum of 370 p.p.m., the mean being 290 p.p.m. The total dissolved solids corresponding to this would vary approximately from a minimum of 450 p.p.m. to a maximum of 780 p.p.m., the mean being 580 p.p.m.

The diversion tunnels at Boulder Dam were closed on February 1, 1935, and water has been stored continuously ever since. The maximum reservoir content to date has been 24,350,000 acre-ft. which occurred early last summer. Cooperative studies have been made at Lake Mead of the temperature and saline variation with depth, which have produced some very interesting results. During the summer months the temperature difference between water at

the surface of the reservoir and at a 425-foot depth is as much as 30° F., and the difference in the total hardness of the two levels is as much as 135 p.p.m. This condition is not uniform throughout the year, however, and the reservoir has not been in operation long enough to indicate what might be considered normal variation. Each winter a definite mixing takes place but the air and water temperatures are not low enough for this mixing to produce a rapid overturn in the reservoir. Instead, the blending of the top and bottom waters is so gradual as not to be noticed except by means of the regular depth samples which are obtained and analyzed.

This stratification indicates that while the water in the reservoir may approach the average which has been estimated, the water discharged into the river below the dam may at times be materially different from this average. This is particularly likely, as gates are provided for withdrawing water from the reservoir only at elevations 900 and 1050, that is, 253 ft. or 403 ft. above normal stream bed. In operating the reservoir an attempt is being made to discharge the better water in the summer when the domestic and irrigation demand is greatest, and to discharge the harder water in the winter when demand is at a minimum. Of course, over a long period of time the average quality of all the water discharged will be the same as the average quality of the inflow, corrected for reservoir evaporation.

After the water leaves Lake Mead it flows downstream to Lake Havasu, through the aqueduct to Cajalco Reservoir, and then by pressure line to the softening and filtration plant. It is estimated that due to the evaporation and leaching action in the concrete lines, the total hardness of the water as it reaches the softening plant may be about 5 per cent greater than it is as it leaves Lake Mead. Table 1 shows the estimated average quality of the water as it will enter the softening and filtration plant, based on the long-time record.

Softening Methods

As soon as the preparation of plans and specifications for the water softening and filtration plant had been authorized, studies were made to determine the method of softening best suited to Colorado River water. Not only did the process selected have to be well adapted to the treatment of aqueduct water, but it had to fit in with other conditions as well.

The zeolite, lime-soda ash, lime-zeolite, and other water softening

processes were studied before a decision was finally made as to the best process to use. Straight zeolite treatment would have completely eliminated the sludge problem but it was rejected for the following reasons: (1) It was too expensive, (2) the sodium content of the water would have been too high, and (3) too much valuable wash and rinse water would have been wasted. Several barium compounds

TABLE 1

Average Quality of Raw Colorado River Water as Delivered to the Softening and Filtration Plant

Chemical Constituents	*parts per million*
Calcium	81.
Magnesium	25.
Sodium and Potassium	87.
Bicarbonate	182.
Sulfate	243.
Chloride	60.
Nitrate	2.8
Iron	0.25
Silica	19.
Fluorine	0.3
Boron	0.15
Total dissolved solids	609.
Hardness as $CaCO_3$	
Total	305.
Carbonate	145.
Non-carbonate	160.
Free carbon dioxide	6.
Hydrogen ion concentration (pH) 8.0	

were suggested as softening reagents but, while they would have been very effective in reducing both the hardness and the sulfate content of the water, they were eliminated from consideration because of their cost, small softening value per unit of weight, and possible poisonous effects. The lime-soda ash and lime-zeolite processes were the ones given most serious consideration.

The lime-soda ash process, as developed for the softening of hard magnesium waters such as Colorado River water, is familiar to most water works chemists. Sufficient lime is added to the water to (1)

neutralize the free carbon dioxide, (2) convert the bicarbonate to normal carbonates, (3) precipitate the magnesium as $Mg(OH)_2$, and (4) raise the pH of the water high enough to complete the precipitation of $Mg(OH)_2$. Sufficient soda ash is added to the water to reduce the non-carbonate hardness sufficiently to produce the hardness desired in the finished water. The excess lime is neutralized, usually, with carbon dioxide gas. This is often called the excess lime-soda ash process, on account of the excess amount of lime used, beyond that required for the chemical reaction, in order to raise the pH of the water.

The procedure in the lime-zeolite method of softening is as follows: (1) Lime is added to the water in just sufficient quantity to neutralize the free carbon dioxide and the calcium carbonate; (2) a portion of the lime-softened water, after filtration and possible pH adjustment, is softened to zero hardness by passing it through a bed of zeolite; and (3) a mixture of the lime-softened water and the lime-zeolite-softened water is then made in proper proportions to give a water of the desired hardness.

A study of the water analysis given in Table 1 shows that all the carbonate hardness of Colorado River water is due to calcium bicarbonate. When just enough lime is added to the water to neutralize the free carbon dioxide and the calcium bicarbonate, the reaction goes to completion rapidly and very pure calcium carbonate is precipitated. The higher the available calcium oxide content of the lime used, the higher will be the purity of the calcium carbonate precipitate. Due to the fact that the calcium carbonate is precipitated free from flocculent magnesium hydroxide, the precipitate is crystalline in structure, is heavy and settles rapidly, leaving a very clear supernatant.

All of the lime used in the lime-zeolite process is effective in reducing the hardness of the water. A large part (almost one-half) of the lime used in the lime-soda ash process accomplishes no softening and is useful only in removing the magnesium and thereby making possible the reduction of the non-carbonate hardness originally caused by the magnesium by means of soda ash. In the lime-zeolite process salt is used (indirectly) for reducing the non-carbonate hardness in place of the more expensive soda ash used in the lime-soda ash process. Therefore, from an inspection of Table 2 and considering major chemical costs only, the lime-zeolite process is less expensive for softening Colorado River water than is the lime-soda ash process.

All of the data on the choice of processes are based on softening average Colorado River water of the analysis given in Table 1. The comparative cost of softening is based on a price of 88 per cent available CaO lime of $5.50 per ton, for soda ash of $16.50 per ton, and for salt of $5.00 per ton. These are the prices, delivered at the

TABLE 2

Comparative Cost of Softening Chemicals

SOFTENING REAGENT	CONSTITUENT TO BE REMOVED OR REDUCED AND QUANTITY IN P.P.M.		REAGENT REQ'D		CHEMICAL COST PER M.G.
			Lb. per m.g. (100% reagent)	Cost per ton*	
	Lime-Soda Ash Process				
Lime	Free carbon dioxide as CO_2	6	63.4	$6.25	$0.20
Lime	Carbonate hardness as $CaCO_3$	145	677.6	6.25	2.12
Lime	Magnesium as Mg	25	480.7	6.25	1.50
Lime	Excess to raise pH as $CaCO_3$	35	166.9	6.25	0.52
	Non-carb. hardness as $CaCO_3$	160			
Soda Ash	Non-carb. hardness removed as $CaCO_3$	110	973.5	16.50	8.03
Total cost of lime and soda ash					$12.37
	Lime-Zeolite Process				
Lime	Free carbon dioxide as CO_2	6	63.4	$6.25	$0.20
Lime	Carbonate hardness as $CaCO_3$	145	677.6	6.25	2.12
	Total hardness after lime softening as $CaCO_3$	195			
Salt	Hardness removed by zeolite as $CaCO_3$	110	2248.0†	5.00	5.62
Total cost of lime and salt					$7.94

* Cost per ton of 100 per cent CaO lime on basis of $5.50 per ton for 88 per cent CaO lime.

† Salt requirement 0.35 lb. per 1,000 grains of hardness removed.

softening plant, which have been bid or quoted the District. Table 2 gives the cost of softening chemicals when using the lime-soda ash and the lime-zeolite processes for reducing the total hardness of the water to 85 parts per million. This hardness is picked arbitrarily and may or may not be the hardness to which the water will be reduced, but regardless of the degree of softening, the comparative

costs of the lime-zeolite and the lime-soda ash processes remain about the same.

A further analysis of the data in Table 2 shows that for soda ash to compete with salt at $5.00 per ton, it would have to cost $7.39 per ton or less, delivered.

The water softening processes may, of course, involve other chemicals such as coagulant, carbon dioxide, chlorine, ammonia, phosphate, activated carbon, etc. Approximately the same quantities of all of these reagents would be required for either process, however, with the exception of carbon dioxide. Since the water is not overtreated with lime, less carbon dioxide will be required for the lime-zeolite process.

Sludge Disposal

One of the principal problems to be solved in designing the plant was the disposal of the sludge produced by the water softening reaction. The quantity of sludge produced by a plant of this size may be tremendous, depending upon the process used, and there are no large streams in the vicinity into which it might be discharged and carried away. The best means of disposal available seems to be lagooning of the sludge on low-value land and handling the material in such a way as to make it unobjectionable. It is also important that the quantity to be disposed of be kept as low as possible.

Sludge is produced by both processes but much less is produced by lime-zeolite. The initial plans do not include facilities for the production of lime from reclaimed sludge, but ultimately it is planned to install equipment for that purpose. In either process, the quantity of sludge to be disposed of will be less when lime reclamation facilities are installed. By using the lime-zeolite process, however, the sludge disposal problem is materially reduced. Table 3 shows the quantity of sludge which will be produced by each process and the amount to be disposed of both with and without lime reclamation.

In the paper entitled "Reclamation and Re-use of Lime in Water Softening" by Mr. Aultman (3), two processes, the Estabrook and the Hoover, were described for producing a water softening sludge sufficiently free from magnesium as to be satisfactory for calcining to lime. Both processes are modifications of the lime-soda ash process and the installation of either increases the cost of a lime-soda ash plant. The lime-zeolite process offers another method of producing magnesium-free calcium carbonate sludge which can be calcined and re-used in the plant.

Comparison of Plant Costs

Estimates and quoted prices show that under the prevailing conditions the cost of a lime-zeolite plant is considerably less than the cost of a lime-soda ash plant of equal capacity incorporating the features of the Estabrook or Hoover process. The cost is less because: (1) Required storage capacity and handling facilities for chemicals are smaller and therefore less expensive; (2) a smaller number of lower capacity chemical feeders are required, (3) but one-third to two-fifths of the total detention time is required in the flocculating and settling basins, (4) fewer flocculating and sludge removal devices are required, (5) the required capacity of the entire carbonation equipment is less, (6) sludge disposal requirements are materially reduced, (7) if used, lime reclamation facilities can be of reduced capacity, and (8) the cost of zeolite softening facilities is less than the above savings.

TABLE 3

Sludge Quantities—plant operating at rate of 100 m.g.d.

	LIME-SODA ASH PROCESS	LIME-ZEOLITE PROCESS
Total sludge produced	4,056 lb. per m.g.	2,495 lb. per m.g.
Sludge required for lime reclamation	2,756 lb. per m.g.	1,471 lb. per m.g.
Excess sludge to waste	1,300 lb. per m.g.	1,024 lb. per m.g.

Table 2 shows that the lime-soda ash process requires about equal volumes of both lime and soda ash, assuming that in storage, lime weighs 50 lb. per cu.ft. and soda ash 35 lb. per cu.ft., and that the lime requirement is about twice that of the lime-zeolite process. The storage capacity for dry chemicals would therefore have to be almost four times as great for lime-soda ash treatment as for lime-zeolite. Adequate facilities would also have to be provided for mechanically handling the increased quantities of chemicals. Offsetting this to a certain extent is the large volume of storage space required for salt with the lime-zeolite process. Salt storage space, however, is relatively inexpensive and handling costs very little. The salt is dumped directly into the storage tanks from trucks or hopper bottom cars, is stored partially submerged in saturated brine, and is rehandled by pumping directly into the zeolite softeners.

The lime feeding and slaking machines required for the lime-soda

ash process would have to have almost twice the capacity of those required for the lime-zeolite process. In addition to the lime feeders, an equal number of feeders and dissolvers of about the same capacity would have to be provided for soda ash. At times when water of maximum hardness must be softened, the soda ash requirement to produce 85 p.p.m. water would reach 1,947 lb. per million gallons.

Tests made on Colorado River water indicate that a flocculating period of one-half hour and a settling period of two hours will give good softening results. Using the Estabrook lime-soda ash process, these flocculating and settling times would be required for three separate steps: (1) in the magnesium removal unit for 25 per cent of the water, (2) in the primary basins for all of the water, and (3) in the secondary basins for all of the water. Three separate steps of flocculating and settling, each with the above detention periods for 100 per cent of the water, would be required for the Hoover process. The lime-zeolite process requires but one, one-half-hour flocculating period, and one, two-hour settling period.

Carbonation equipment must have sufficient capacity to neutralize the excess lime used in the lime-soda ash process and, if desirable, to reduce the pH of the water going to the filters. In the lime-zeolite process, carbonation is used for pH correction only. The lime-soda ash process requires at least twice as much carbonation capacity as the lime-zeolite process.

Table 3 shows that with lime reclamation the quantity of sludge which must be disposed of is 1.3 times as great for the lime-soda ash process as for the lime-zeolite process; and 1.6 times as great without lime reclamation.

Since lime-zeolite treatment requires only about half as much lime as lime-soda ash treatment, lime reclamation equipment need be only about half the capacity for the former as for the latter. This is true of all of the equipment necessary for lime reclamation, including sludge pumps, sludge filters, lime kilns, lime cooling equipment, conveying equipment, etc. The size of the building housing the equipment would also be reduced in proportion.

Estimates made by the District's engineers indicate that the reduced cost of a lime-zeolite plant below the cost of a lime-soda ash plant, more than offsets the cost of installing zeolite softeners. This, of course, applies to the plants only and does not include a drain line for the disposal of waste brine which is required for the zeolite plant.

The waste brine and rinse water from the regeneration of the

zeolite softeners cannot be disposed of on the plant site or in any of the water courses in the vicinity because of possible contamination of ground water. A drain line is therefore required extending from the plant site to tidewater and of sufficient capacity to carry the maximum quantity of waste brine and rinse water. The cost of this line makes the initial lime-zeolite plant cost more than an initial lime-soda ash plant, but the cost of the two plants of ultimate capacity (400 m.g.d.) shows a saving in favor of lime-zeolite.

To aid in making a decision as to the best method of treatment, studies were made as to the availability of salt and soda ash in the tonnages which would be required for the initial and ultimate plants. The investigations indicated that salt was available in unlimited quantities from a number of near-at-hand sources, while soda ash might become difficult and more expensive to obtain when the plant approached ultimate capacity. The soda ash produced in southern California comes from natural sources and it appears that economical production of greatly increased quantities might depend upon the producer's ability to market increased outputs of potash, borax, and other related chemicals which occur in conjunction with soda ash.

Plant Design

After careful consideration of all of the elements as outlined, the lime-zeolite process was selected as most economical and best adapted to the treatment of Colorado River water. The plant was accordingly designed to fit that process. Much of the equipment has already been purchased.

The design of the plant was complicated by the fact that its intake may be said to be located at the Cajalco Reservoir, 36 miles from the plant site, and with about 36 miles of 116-inch to 152-inch pipe line without overflow spillways or regulating valves between the plant and the reservoir. There can be no regulating valves in the pipe line at the plant because there is a difference in head between the water level in the plant and high water level in the Cajalco Reservoir of approximately 265 ft., and the pipe is not designed to withstand the pressure which would be built up if the flow of water were throttled or stopped at the plant. The plant, therefore, had to be designed so that water entering the aqueduct at the Cajalco intake, at any given rate between zero and maximum flow, must flow unobstructedly through the plant, be properly treated, and flow back into the aqueduct. In order to treat all of the water correctly, all of the chemical

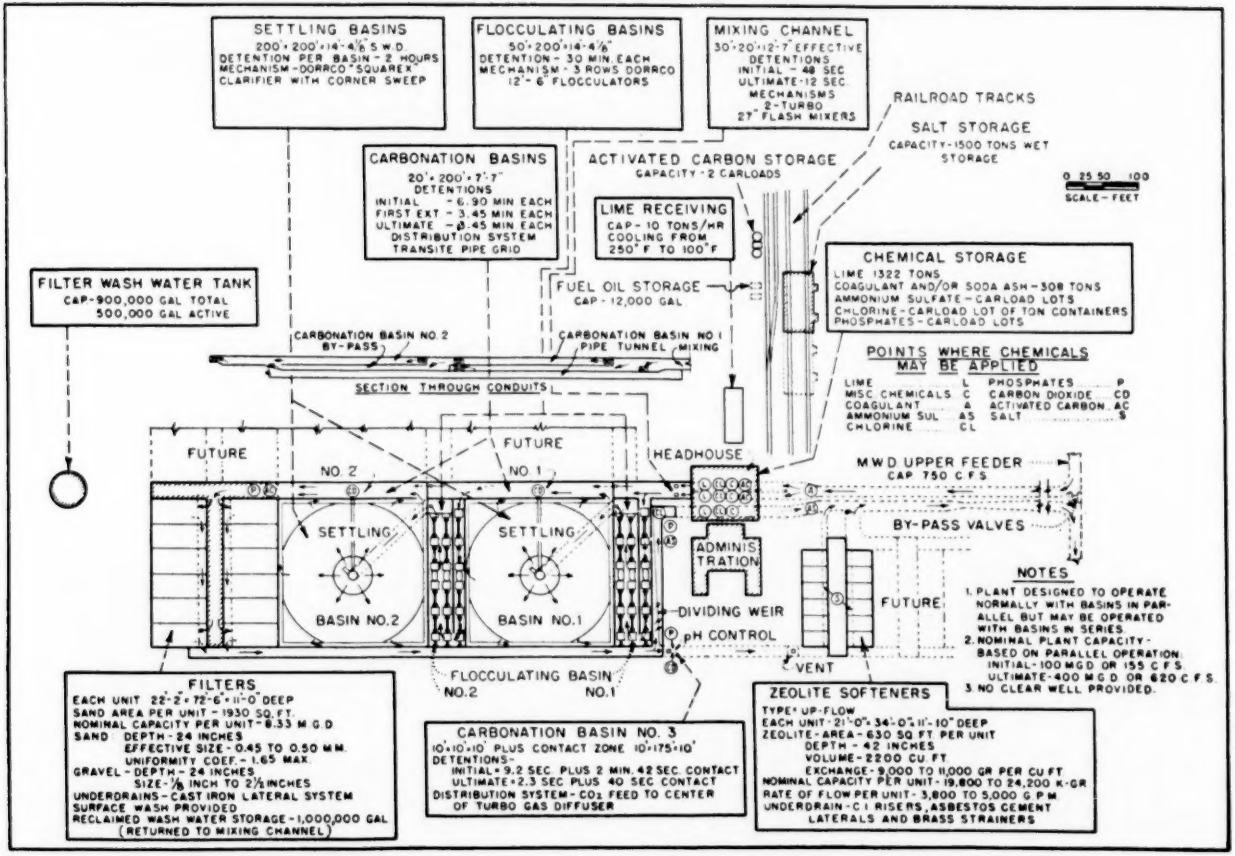
SETTLING BASINS
200'×200'×14-4½" S.W.D.
DETENTION PER BASIN - 2 HOURS
MECHANISM - DORRCO "SQUAREX"
CLARIFIER WITH CORNER SWEEP
FLOCCULATING BASINS
50'×200'×14-4½"
DETENTION - 30 MIN. EACH
MECHANISM - 3 ROWS DORRCO
12' - 6" FLOCCULATORS
MIXING CHANNEL
30'×20'×12'-7" EFFECTIVE
DETENTIONS
INITIAL - 48 SEC.
ULTIMATE - 12 SEC.
MECHANISMS
2 - TURBO
27" FLASH MIXERS
RAILROAD TRACKS
SALT STORAGE
CAPACITY - 1500 TONS WET STORAGE
CARBONATION BASINS
20'×200'×7'-7"
DETENTIONS
INITIAL - 6.90 MIN EACH
FIRST EXT - 3.45 MIN EACH
ULTIMATE - 3.45 MIN EACH
DISTRIBUTION SYSTEM
TRANSITE PIPE GRID
ACTIVATED CARBON STORAGE
CAPACITY - 2 CARLOADS
0 25 50 100
SCALE - FEET
LIME RECEIVING
CAP - 10 TONS/HR
COOLING FROM
250° F TO 100° F
CHEMICAL STORAGE
LIME 1322 TONS
COAGULANT AND/OR SODA ASH - 308 TONS
AMMONIUM SULFATE - CARLOAD LOTS
CHLORINE - CARLOAD LOT OF TON CONTAINERS
PHOSPHATES - CARLOAD LOTS
FILTER WASH WATER TANK
CAP - 900,000 GAL TOTAL
500,000 GAL ACTIVE
FUEL OIL STORAGE
CAP - 12,000 GAL
CARBONATION BASIN NO. 2
BY-PASS
CARBONATION BASIN NO. 1
PIPE TUNNEL MIXING
SECTION THROUGH CONDUITS
POINTS WHERE CHEMICALS MAY BE APPLIED
LIME L
MISC. CHEMICALS C
COAGULANT A
AMMONIUM SUL AS
CHLORINE CL
PHOSPHATES P
CARBON DIOXIDE CD
ACTIVATED CARBON AC
SALT S
FUTURE
NO. 2
FUTURE
NO. 1
HEADHOUSE
MWD UPPER FEEDER
CAP 750 C.F.S.
SETTLING
BASIN NO. 2
SETTLING
BASIN NO. 1
ADMINISTRATION
BY-PASS VALVES
DIVIDING WEIR
pH CONTROL
FUTURE
VENT
FLOCCULATING BASIN
NO. 2
NO. 1
NOTES
1. PLANT DESIGNED TO OPERATE NORMALLY WITH BASINS IN PARALLEL BUT MAY BE OPERATED WITH BASINS IN SERIES.
2. NOMINAL PLANT CAPACITY - BASED ON PARALLEL OPERATION
INITIAL - 100 MG D OR 155 C F S
ULTIMATE - 400 M G D OR 620 C F S
3. NO CLEAR WELL PROVIDED.
FILTERS
EACH UNIT 22'-2"×12'-6"×11'-0" DEEP
SAND AREA PER UNIT - 1930 SQ. FT.
NOMINAL CAPACITY PER UNIT - 8.33 M G D
SAND DEPTH - 24 INCHES
EFFECTIVE SIZE - 0.45 TO 0.50 M.M.
UNIFORMITY COEF. - 1.65 MAX.
GRAVEL - DEPTH - 24 INCHES
SIZE - ⅛ INCH TO 2½ INCHES
UNDERDRAINS - CAST IRON LATERAL SYSTEM
SURFACE WASH PROVIDED
RECLAIMED WASH WATER STORAGE - 1,000,000 GAL
(RETURNED TO MIXING CHANNEL)
CARBONATION BASIN NO. 3
10'×10'×10' PLUS CONTACT ZONE 10'×175'×10'
DETENTIONS -
INITIAL - 9.2 SEC. PLUS 2 MIN. 42 SEC. CONTACT
ULTIMATE - 2.3 SEC. PLUS 40 SEC. CONTACT
DISTRIBUTION SYSTEM - CO₂ FEED TO CENTER OF TURBO GAS DIFFUSER
ZEOLITE SOFTENERS
TYPE - UP-FLOW
EACH UNIT - 21'-0"×34'-0"×11'-10" DEEP
ZEOLITE - AREA - 630 SQ. FT. PER UNIT
DEPTH - 42 INCHES
VOLUME - 2200 CU. FT.
EXCHANGE - 9,000 TO 11,000 GR. PER CU. FT.
NOMINAL CAPACITY PER UNIT - 19,800 TO 24,200 K-GR.
RATE OF FLOW PER UNIT - 3,800 TO 5,000 G.P.M.
UNDERDRAIN - C.I. RISERS, ASBESTOS CEMENT LATERALS AND BRASS STRAINERS

FIG. 1. Flow Diagram and Design Data of Water-Softening Plant

feeding devices will feed in proportion to the flow of water into the plant, the flow rate through the filters will be automatically regulated to equal the rate of flow of water into the plant, and the proportion of flow through the zeolite softeners will be automatically adjusted as the rate of flow into the plant varies.

Figure 1 is a general layout of the plant showing, by means of arrows, the path of the water through the plant and, by means of symbols, the points of application of chemicals. On Fig. 1 are also shown the essential design data of the various units.

The plant will have an initial capacity of 100 m.g.d. and an ultimate capacity of 400 m.g.d., provision being made for expansion in 100 m.g.d. units. Certain features in the initial plant will be built large enough for the ultimate development because of the impossibility or inadvisability of building those units with provision for future expansion. The various units of the plant which have been designed for ultimate capacity, half capacity, and initial capacity, are as follows:

A. Structures and equipment designed for ultimate capacity:
   1. Turnout structure in main aqueduct
   2. Influent and effluent conduits and meters
   3. Head house
   4. Chemical feeders
   5. Chemical handling equipment
   6. Administration building
   7. Flash mixers
   8. Filtered water effluent conduits
   9. Wash water reclamation basin and pumps
   10. Lime unloading building
   11. Activated carbon feed and storage
   12. Railway siding and drives (approximately complete)
   13. Brine disposal line

B. Structures designed for half capacity:
   1. Conduits to and from mixing and settling basins
   2. Filter wash water tank

C. Structures designed for initial capacity:
   1. Flocculating basins
   2. Settling basins
   3. Filters
   4. Zeolite softeners
   5. Salt storage facilities

FIG. 1. Flow Diagram and Design Data of Water-Softening Plant

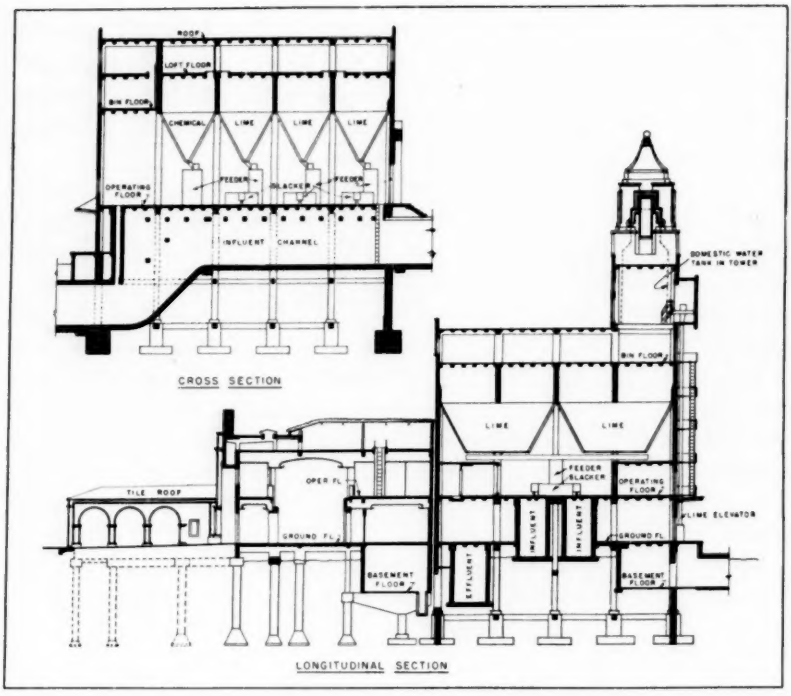

FIG. 2. Sections of Administration Building and Head House

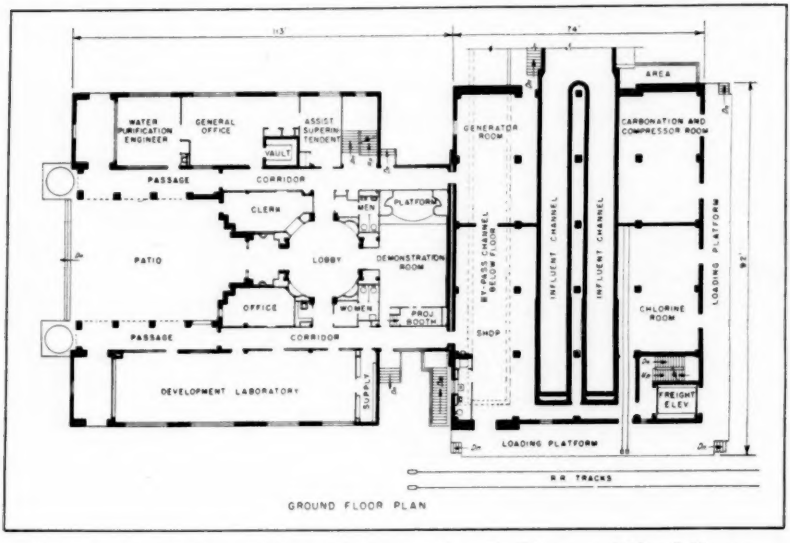

FIG. 3. Ground Floor Plan, Administration Building and Head House

The raw water will be carried through the head house in two channels as shown in Fig. 2, either of which will carry the full capacity of the ultimate plant. Two channels are provided so that either can be closed off, drained, and cleaned without interfering with the operation of the plant. The chemical dry feed machines will be located on the operating floor directly over and discharging into the raw water conduits, so that difficulties due to clogging of chemical lines will be eliminated.

As shown in Figs. 1, 2, 3, and 5, all of the chemical storage and feeding equipment will be located in the head house with the excep-

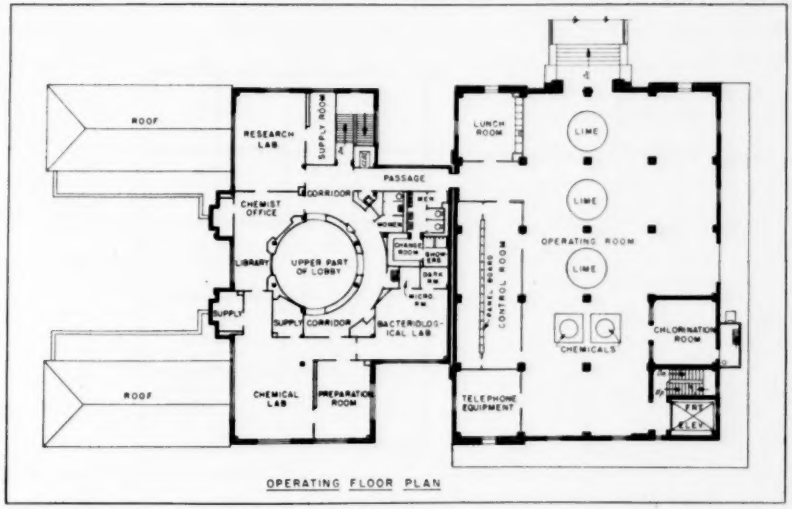

FIG. 4. Operating Floor Plan, Administration Building and Head House

tion of activated carbon and salt for zeolite softener regeneration. All of the chemical feeders will be electrically controlled and will operate in step with the raw water meters, feeding in direct proportion to the rate of flow of raw water into the plant. Three gravimetric dry feeders and slakers will be provided for lime and two gravimetric feeders with dissolving tanks will be provided for coagulant, soda ash, or other chemicals which it may be desirable to use in the future. Phosphate and ammonium sulfate will be fed in solution from dissolving tanks located on the third floor of the head house. Carbon dioxide will be generated initially by burning natural gas or oil and ultimately, if lime reclamation facilities are installed,

from the lime kiln stack gases. In either case, compressors will be used for pumping the gas into the water. For chlorination, standard type chlorinators will be used.

Particular care has been taken to keep activated carbon away from the main plant. Upon delivery, carbon will be dumped immediately into one of two storage tanks, each having a capacity of one carload, and made up into a water suspension of known strength. The tanks will be equipped with dust removal equipment for use while filling, agitators for keeping the carbon in suspension, and meter controlled feed pumps for pumping the suspension into the water at any desired rate. The tanks will be located approximately

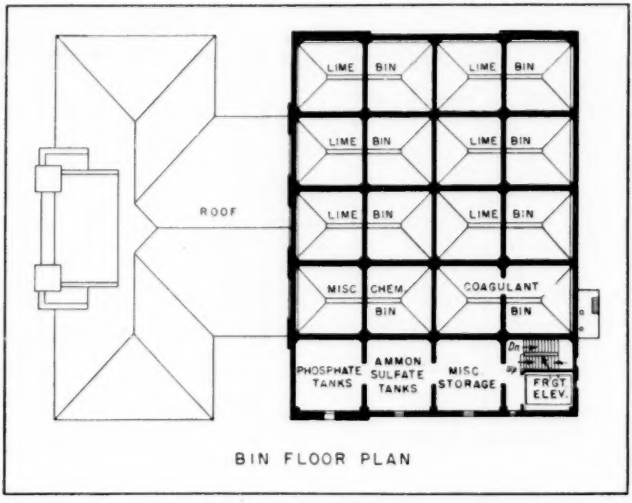

FIG. 5. Bin Floor Plan, Administration Building and Head House

300 ft. away from the head house and its is believed that the usual carbon nuisance will be eliminated.

The flash mixers, flocculators, and sludge removal mechanisms have already been purchased and will be installed in the basins shown in Fig. 1. Due to the heavy, crystalline nature of the calcium carbonate precipitate, the flocculators and sludge removal mechanisms have been designed with extra strength and heavier drives. The flocculating basins have been designed for longitudinal flow because it is felt that this gives more complete use of the detention time and less likelihood of short-circuiting. The settling basins will have center feed and peripheral take-off over an adjustable steel

weir plate. Each pair of flocculating and settling basins is rated at 50 m.g.d. based on one-half-hour flocculation and two-hour settling periods. From laboratory and small scale tests, however, the indications are that this rate can be exceeded materially due to the rapid rate of reaction and the quick settling properties of the calcium carbonate precipitate.

The filters are of conventional design with perforated pipe underdrains, concrete wash water troughs, hydraulically operated control valves, pipe lateral type surface wash facilities, and individual rate of flow controllers with master control. The master control will operate to keep the rate of flow through the filters in step with the rate of flow into the plant, the controller valves all opening or closing together in equal amounts as the influent flow increases or decreases. Rate of flow controllers will also be placed in the surface wash and backwash mains.

Wash water will be stored in a steel standpipe located north of the plant and will flow to the filters by gravity. Used wash water will flow to basins underneath the filters and will be reclaimed by pumping it back to the main water conduit ahead of the flash mixers.

After lime softening and filtration, the water will be divided into two portions, one portion flowing back into the aqueduct through a conduit located beneath the head house (Figs. 1 and 2), where the pH can be corrected if necessary; the other portion flowing through zeolite softeners where the hardness of that portion will be reduced to zero and thence back into the aqueduct. Enough water will be passed through zeolite to reduce the hardness of the combined waters to the desired figure.

The zeolite softeners will operate upflow, that is with the water entering the bottom of the softener and flowing up through the bed of zeolite. Upflow operation reduces the head required to force the water through the bed and reduces the quantity of waste water by eliminating the bed washing operation necessary when downflow is used. Brining and rinsing will be downflow.

There will be 12 units of ample softening capacity to reduce 100 m.g.d. of water of the highest anticipated hardness to 75–85 p.p.m. with two units out of service. Each unit will contain approximately 2,200 cu.ft. of zeolite. Operating rates will range between 6 and 8 gallons per sq.ft. per min.

The design is based on the use of synthetic gel type zeolite with an operating exchange value of 9,000 to 11,000 grains of hardness

per cu.ft. of zeolite, and a salt consumption of not to exceed 0.35 lb. per 1,000 grains of hardness removed. All of the commercially available zeolites were considered but the synthetic material seemed best for the treatment of Colorado River water.

Both greensand and processed greensand would give satisfactory softening results but due to their lower exchange value (2,800 to 5,500 grains per cu.ft.) the quantity of water softened between regenerations would be less than for the gel type and consequently, the percentage of rinse water would be increased. Greensands of either type, due to their high specific gravity, are difficult to operate upflow with good exchange value, and, if operated downflow, wash water would have to be used for breaking up the bed before regeneration and more head would be lost in the operation of the softeners. If greensand were used in place of synthetic zeolite, in order to obtain the same exchange capacity and length of softener runs, approximately twice the volume of processed greensand would be required and four times the volume of ordinary greensand. As a consequence, the entire zeolite plant would have to be very materially increased in size.

Carbonaceous zeolite, operating on the sodium cycle, and being regenerated with salt, would give satisfactory results. Its exchange capacity, however, is lower than gel type synthetic zeolite and due to its being extremely light, it can be operated upflow only at reduced rates without carrying over into the finished water.

Carbonaceous zeolite can also be operated on a hydrogen cycle in which case it is regenerated with a strong acid. Water passed through a bed of this mineral operating on the hydrogen cycle will have all of the cations replaced by hydrogen and the water will contain free acid. By this process carbonates and bicarbonates can be almost completely removed by aerating the acid water. This is the first step in a recently developed process of completely demineralizing water. For the second step a new type of base exchange material is used which has the property of removing the anions from acid waters, such as are produced by the first step, and replacing them with hydroxyl ions, thus producing a mineral-free water. When exhausted the material is regenerated with alkali hydroxides or carbonates. If economically feasible this process could be used to demineralize completely a portion of the water and mix this with the correct amount of raw water to give the hardness desired in the finished product. To date, however, this process has but little

background of experience, the life of the material is unknown, the quantity of rinse water required is excessive and the chemical cost is high.

Both greensand and carbonaceous zeolites are possibly more resistant to attack then gel type zeolites, but the conditions under which the gel type might disintegrate are well known and, when used with the right kind of water, this mineral is both long-lived and economical to use. The lime treated, filtered, and chemically controlled water which goes to the softeners will be properly conditioned for softening with the synthetic gel type mineral.

Due to the high exchange value, the amount of water softened between regenerations will be a maximum, using the gel type zeolite. The amount of rinse water to be wasted will therefore be a minimum. It is important that the quantity of water wasted be kept at a minimum for two reasons: (1) Water which has been brought from the Colorado River and elevated 1,617 ft. is too valuable to throw away needlessly; and (2) the greater the quantity of water wasted, the larger and more expensive will be the waste line to tidewater. With gel type zeolite the percentage of water wasted when softening average water to 85 p.p.m. total hardness is calculated to be approximately 1.15 to 1.40 per cent of the total flow through the entire plant.

The zeolite softeners will be semi-automatic in operation. Manually operated four-way valves on the operating tables will control the opening and closing of the raw water influent and soft water effluent valves. Positive interlocks will be provided which will make it impossible to begin the regeneration cycle if either or both of these valves are even slightly open, or to open either of these valves if the regeneration process is in progress. It should therefore be impossible for a careless operator to get brine into the distribution system. After the influent and effluent valves have been closed, the regeneration operation will be initiated by push-button control. Regeneration then proceeds automatically to completion, leaving the softener in readiness to be placed back in operation manually.

Brine reclamation facilities will be installed in connection with the zeolite regenerating equipment and the automatic devices will be so arranged that the softeners can be regenerated either with or without the reclaiming and re-use of the partially spent brine.

The saturated brine from the salt storage tanks will be filtered through a battery of three pressure filters before it is used for re-

generation. It is felt that by filtering the brine the life of the zeolite may be increased and less expensive but slightly dirty salt may be used.

Offices and laboratory facilities are centralized in the administration building. In the offices the water treatment records for all of the District operations will be tabulated, studied, and filed, and all routine office work will be performed. The laboratories will be ample in size, well lighted, and equipped for all kinds of chemical, bacteriological, and biological work incident to water purification practice. A development laboratory where new developments, processes, etc., can be tried out on small plant scale, will occupy one wing of the administration building. A demonstration room, seating about 70 people, is also provided, to accommodate those groups of students and visitors who, as experience has shown, visit all of the larger plants in numbers up to 5,000 to 6,000 per year. This room will be equipped with chemical and bacteriological demonstration facilities and with a projection room for sound or silent moving pictures. The administration building and some rooms in the head house will be air-conditioned.

The Colorado River Aqueduct construction is under the direction of F. E. Weymouth, General Manager and Chief Engineer, and Julian Hinds, Assistant Chief Engineer. The water-softening investigations were under the supervision of William W. Aultman, Engineer. Consulting Engineers Charles P. Hoover and James M. Montgomery of Columbus, Ohio directed the investigations and the preparation of plans and specifications for the water-softening and filtration plant.

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Silting of Reservoirs

By L. Standish Hall

THE problem of silting of reservoirs is one that has perplexed the hydraulic engineer since the first dam was constructed. The great increase in the number of reservoirs used for municipal water supply, power, and irrigation has emphasized the need for more knowledge of the rates of silting in various reservoirs already constructed and of the methods of retarding the rate of deposition of silt so as to prolong the useful life of these structures.

This paper is divided into two parts: first, a description of the results of measurements of silt deposits in various reservoirs owned by the East Bay Municipal Utility District with comparison of rates of silting in other reservoirs; and second, an outline of the proposed methods of controlling the rate of erosion on the District's watersheds. Although the Mokelumne River is the main source of water supply for the East Bay Cities, the principal problem of erosion control is located above the local reservoirs on the watershed lands near the East Bay Cities, situated in valleys lying between the parallel ridges easterly of the municipal area.

The total drainage area of the local watersheds, including the area adjacent to the proposed Pinole Reservoir, is approximately 86 sq.mi. The various drainage basins are adjacent to one another so that the entire watershed area has an approximate length of 18 mi. and an average width of $4\frac{3}{4}$ mi. The streams draining this area include San Pablo Creek, San Leandro Creek, Pinole Creek and Lafayette Creek. The drainage areas on the individual streams above existing or proposed dam sites are as follows: San Leandro Creek, 42.05 sq.mi.; San Pablo Creek, 32.15 sq.mi.; Lafayette Creek, 1.34 sq.mi.; and Pinole Creek, 10.50 sq.mi. The total is 86.04 sq.mi.

A paper presented on October 26, 1939, at the California Section Meeting at San Francisco, by L. Standish Hall, Hydraulic Engineer, East Bay Municipal Utility District, Oakland, Calif.

There are two reservoirs on San Leandro Creek: Lake Chabot and Upper San Leandro Reservoirs, the latter having 30.30 sq.mi. above the Upper San Leandro Dam and the remaining 11.75 sq.mi. being between this dam and the Chabot Dam.

The reservoir capacities at spillway level of the individual reservoirs, according to the most recent surveys, are as shown in Table 1.

The topography adjacent to all of these reservoirs is mainly hilly and mountainous except in small detached valleys and along minor stream bottoms. The elevations range from a minimum of about 240 ft. above mean sea-level at Chabot Reservoir to a maximum of over 2,000 ft. above mean sea-level at Rocky Ridge in the San Leandro Watershed. Several peaks scattered throughout the area have elevations ranging between 1,500 and 2,000 ft. above mean sea-level.

TABLE 1

Reservoir Capacities

RESERVOIR	CAPACITY	
	Acre-Ft.	Million Gallons
Chabot	12,553	4,090
Upper San Leandro	41,398	13,490
San Pablo	43,149	14,060
Lafayette	4,450	1,450
Pinole (proposed)	33,100	10,786
Total	134,650 acre-ft.	43,876 million gallons

The rock formation consists principally of poorly consolidated sandstone and shale rocks; although, there are small outcrops of volcanic rocks and also some indurated sedimentary formations. The soils derived from the underlying rock formations are principally residual soils, with small strips of transported soils following along the stream channels. Nearly all of the soils throughout the drainage areas are of heavy texture (clay and clay loams) with only small areas of medium-textured soils (silt loam or fine sandy loam).

The rainfall throughout this territory ranges from a minimum of 21.5 in. at Lake Chabot to a maximum recorded precipitation of 30.3 in. in the Upper San Leandro Basin. The recorded maximum may be exceeded in the vicinity of some of the higher peaks, but no actual rainfall records are available. The seasonal distribution of

the rainfall results in practically no precipitation during the summer months, with heavy rains during the winter and spring periods, 89.0 per cent of the precipitation coming between November and April. The average distribution at Lake Chabot, which can be accepted as typical of the area, is shown in Table 2.

A study of maximum daily rainfall intensities at precipitation stations on the watersheds indicates that once in a period of 25 years a precipitation of $4\frac{1}{2}$ in. in 24 hr. can be expected in the lower belts having the smaller annual precipitation, while 6 in. in 24 hr. can be expected in the same period of years at the higher elevations.

Only small areas of cultivated land exist on these watersheds, located principally in the small valleys and along the stream channels. The hilly portions of the territory are utilized mainly as grass land or are wooded or brush covered. As the District owns 67 per cent

TABLE 2

Average Rainfall Distribution at Lake Chabot

MONTH	PER CENT	MONTH	PER CENT
July	0	January	20.1
August	0.2	February	18.2
Sept.	1.6	March	15.1
Oct.	4.8	April	7.4
November	9.9	May	3.3
December	18.4	June	1.0

Year, 100 per cent.

of the total watershed lands on San Pablo, San Leandro, and Lafayette Creeks, the character of the vegetation will probably remain much the same in the future. The percentage of land owned by the District varies on the different watersheds as follows: Lafayette, 100 per cent; Chabot, 92 per cent; Upper San Leandro, 45 per cent; San Pablo, 56 per cent; and Pinole, 24 per cent.

The slopes of the ground surface draining into the main streams are relatively steep and range from 10 to 40 per cent grades and even higher. Due to the heavy soil, particularly in the hill portions, very little sheet erosion has occurred. Gulleys are quite numerous on the steeper slopes, but the growth of vegetation has partially stabilized this type of erosion in many of the gulleys. Some areas of severe erosion exist, but these represent only a small percentage of the total drainage area. Over the entire hill area, 25 to 75 per cent of the top soil has been removed by erosion.

Mokelumne River Watershed

The total drainage area on the Mokelumne River watershed above Pardee Dam is 575 sq.mi., of which the District owns only $3\frac{1}{2}$ per cent. A good deal of the upper drainage area is unpatented land embraced within the boundaries of the Stanislaus and El Dorado National Forests. The distance from the Pardee Dam to the crest of the divide of the Sierra Nevadas is about 60 mi. The lower portion of the drainage area is very narrow and practically limited to the width of the main river canyon, with the result that for a distance of about 16 mi. above Pardee Dam the drainage area has an average width of 3 mi. East of this point, which is at the junction of the North and South forks, the drainage area widens out, and the easterly 44 mi. has the average width of approximately 12 mi. The erosion from the upper drainage area is partially intercepted by reservoirs belonging to the Pacific Gas and Electric Company. The Salt Springs Reservoir on the North Fork intercepts a drainage area of 160 sq.mi., while the reservoir on Bear River intercepts an area of 28 sq.mi. Deducting this area of 188 sq.mi., the District's reservoir would intercept material eroded from the 387 sq.mi. below the Pacific Gas and Electric Company storage dams. The total capacity of Pardee Reservoir is 210,000 acre-ft. with a dam 358 ft. in height.

The topography of the watershed is mountainous, with the steep-walled Mokelumne Canyon incised in the undulating uplands. From maximum altitudes of over 10,000 ft. at the crest of the range, the entire drainage area slopes to the southwest, following the general tilt of the Sierran block. The rocks underlying the drainage area include shales separated by intrusive dikes of igneous rocks in the foothills, with grano-diorite capped in places with volcanic flows and detritus predominating in the higher mountains. The soils are residual in the main, but with many areas of bare granite and other rocks exposed in the higher mountains.

The mean annual precipitation varies from an average of 23 in. at Pardee Dam to 60 in. on Bear River Ridge, the average precipitation over the watershed being about 43 in. per year. The distribution of the precipitation is quite similar to that previously given for the Coast Range area, the heaviest precipitation occurring during the months of December to March. During these months snowfall

is general in the mountain regions above 4,000 ft. elevation and often at lower levels.

The vegetation consists of deciduous trees and brush in the foothills near Pardee Reservoir, gradually changing to conifers at the higher elevations.

In spite of the steep slopes, erosion is not pronounced, due to the shallow soil and many bedrock exposures.

Reservoir Sedimentation Surveys

Measurement of the silt deposits in the District reservoirs has been made following the procedure used by the Soil Conservation Service of the U. S. Department of Agriculture.

There are two methods of making surveys of reservoirs for the purpose of determining sedimentation deposits. These are known as the Contour Method and the Range Method. The Contour Method is best applied when 50 per cent or more of the original area of the lake is silted and the ground surface on the delta deposits is above the level in the reservoirs. The Range Method, which is made more rapidly than the Contour Method, is used where the deposits are largely below the water level in the reservoir. It also has the advantage that it permits of the actual determination of the thickness of the silt.

In making a survey by the Range Method, the primary control must be established by triangulation, to which all the ranges are tied. This triangulation can be shown on a map varying in scale from 1 in. = 100 ft. to 1 in. = 1,000 ft., depending on the size of the reservoir. The secondary control for use in small bays or arms of the reservoir can be established from magnetic bearings. The ranges should be run across the lake with each subdivision representing equal silting conditions as far as this can be determined in advance. The range line should be parallel wherever possible, but an angle of 10 degrees between separate ranges is not objectionable. A maximum angle of 30 degrees between ranges can be tolerated. Where large bends in the reservoir occur, the ranges should be close together with a maximum angle of 90 degrees between ranges. Any error in the survey is thereby concentrated in a short distance. A new series of ranges should be started for each tributary. The first range should be in the mouth of a tributary and perpendicular to its general direction. If the silting in the tributaries is only

slight, the ranges can be started at the head of the tributary and continued downstream as far as necessary. The location of the ranges should be marked with permanent concrete markers and elevations established. While soundings are being taken, the water surface elevation at the section should be reported hourly by reading of a staff gage. If the reservoir level is fluctuating rapidly, or for sections in the upper end of the reservoir, the temporary gage can be established at the range line.

Elevations of the reservoir bottom below the water surface are taken by soundings, while elevations above the water surface are secured by levels. When taking soundings, intervals of 20 feet are suitable up to ranges 500 ft. in length. On longer ranges or on very regular profiles, intervals up to a maximum of 100 ft. can be used. In taking soundings, an 8-pound cast-iron sounding pea is used. The sounding pea is fastened by a ring to a hame hook attached to a copper or bronze centered sounding line, which has been graduated in feet. Tenths of a foot are estimated.

The boat from which soundings are taken is kept on the range line by means of two range targets set on the shore about 100 ft. apart. The targets consist of a light wooden frame on which white cloth has been tacked. The center strip of red cloth is nailed in the middle of the target. These two targets are mounted on 10-foot poles. By use of these targets the boat can be kept in line for a distance of 1,000 or 2,000 ft. from the targets. On short ranges, and even for distances up to 1,000 ft., distances from shore can be measured on a graduated tag line fastened to the banks, but on longer courses the location of the boat at each sounding can be determined by a transit located on one of the triangulation points.

By use of spuds, the depth of silt can be determined to the nearest tenth of a foot. Two sizes of spuds are used, made from case-hardened steel (similar to axle shafting) into which grooves have been machined at intervals of 0.1 ft. Each groove tapers outward from a maximum depth of $\frac{3}{16}$ in. to zero at the rim of the next 0.1 ft. graduation above. The short spud is 6 ft. in length and $1\frac{1}{2}$ in. in diameter at the rim above each groove. The total weight of this spud is 27 lb. At the top of the rod a hole is drilled, through which a rope is attached. The larger spud is similar in design, but is 10 ft. in length and $1\frac{1}{4}$ in. in diameter. This spud weighs 43 lb. and is used to obtain deeper penetration. In operation, the spud is lowered from the side of the boat in a vertical position until it hits the silt

deposits. In silt that has never been exposed to air and is consequently of very low density, the spud seldom fails from its own weight to penetrate through the silt into the old soil, even though the depth of the deposits is several feet greater than the length of the spud. In determining the depth of the silt deposits, the water depths are first measured and the depth of silt then determined. The elevation of the old reservoir bottom can then be obtained by subtraction.

Spud observations are taken on every second or third sounding on ranges up to 500 ft. in length, and the spacing can be increased to every fifth sounding on longer ranges. The spacing of spud measurements depends largely upon local conditions. The edge of the silt should be determined by probing with the spud, when under water; or by observation, when the silt line is above the water's edge. The spud observations should be taken in conjunction with the soundings and not separately.

When sounding with the spud, it can be lowered slowly over the side of the boat; and if the silt deposits have not been consolidated by drying in the open air, the spud will penetrate entirely through the deposits to the original bottom of the reservoir and into the underlying soil. The boat must be kept stationary by an anchor or guy line when the spud is being used. In deep water the sounding line can be mounted on a reel to facilitate lowering and raising of the spuds. The distinction between the silt deposits and the old soil can be made: (1) by the softness and lack of grit in the silt; (2) by the difference in color between silt and underlying soil; (3) by stiffness of the old soil, if clay; (4) by failure of silt to adhere to the spud, if sandy; and (5) by leaf fragments in the silt and portions of roots in the old soil.

In deep water where the silt tends to wash off while raising the spud back to the boat, one side of the spud can be smeared with cup grease. This is very effective in obtaining samples, especially if the deposits are sandy.

The contour method is best used on deltas at the head of the reservoir. The actual thickness of the silt is not difficult to determine if an old contour map is available. If an original map is not available, extensive borings must be made.

A very complete determination of the rate of silting for Lake Chabot is available as the results of surveys taken to measure the extent of sedimentation at various intervals over the period of 48 years

that the reservoir has been in operation. These surveys were made by the contour method without direct determination of the thickness of the silt deposits. The original survey of reservoir capacity was made in 1875 and resurveys were made in 1900, 1910, and 1923. The total loss of capacity in the 48-year period is 3,307 acre-ft., or 21 per cent of the original capacity. This total silt deposit is slightly less than previously published, due to a correction made for the differences in the base data used in making the various surveys. These discrepancies were disclosed as a result of recent check levels. The data on silting, reservoir capacity, and runoff are given in Table 3.

TABLE 3

Silting of Chabot Reservoir—Drainage Area 42.05 sq.mi.

YEAR OF SURVEY	INTERVAL IN YEARS	CAPACITY AT SPILLWAY NO. 1		SILT DEPOSIT IN RESERVOIR ACRE-FT.	TOTAL RUNOFF FROM DATE OF PREVIOUS SURVEY ACRE-FT.	MEAN ANNUAL RUNOFF IN ACRE FEET FROM DATE OF PREVIOUS SURVEY ACRE-FT.	RATE OF SILTING		
		Million Gal.	Acre-Ft.				Mean Annual Acre-Ft.	Cu.Ft. Per Acre of Drain. Area	Per cent of Total Runoff
1875	—	5169	15,860						
1900	25	4675	14,350	1510	470,300	18,800	60.4	98	0.32
1910	10	4250	13,040	1310	220,770	22.100	131.0	212	0.59
1923	13	4090.5	12,553	487	214,700	16,500	37.5	61	0.23
Total & Means....	48			3307	905,770	18,870	68.9	112	0.37

The Upper San Leandro Reservoir was surveyed in 1935 in cooperation with the Soil Conservation Service of the U. S. Department of Agriculture. The contour method was also used in this survey without direct determination of the silt deposits. The results of this survey are questionable due to the small deposit of silt, which would result in a high percentage of error, due to possible discrepancies between the original and the resurvey. A resurvey of the reservoir by the District is now in progress, using the sounding spud and range method previously described, which will permit of a direct measurement of the thickness of the silt beds. In the 1935 survey it was found that the total sedimentation in the period of 11 years amounted to 170 acre-ft. or 0.4 per cent of the original capacity.

In 1939 a survey was made of San Pablo Reservoir by the range method, using the sounding spud in determining the silt depth. Computations of the cross-sections taken on this survey show that a total of 1,133 acre-ft. of silt had been deposited in the reservoir in 21 years that it has been in service, or that a total of 2.6 per cent of the original capacity has been lost in this period of time.

A sedimentation survey of Pardee Reservoir was made in the latter part of 1938, but the results of this survey are not yet available.

The comparison of the silting in the District Reservoirs with that measured in other reservoirs where similar surveys have been made is compiled in Table 4. These results were taken from various sources, as indicated in the footnote on the table. It will be observed that the rate of silting varies considerably for various reservoirs and a comparison of the rate of silting between the various surveys taken at Lake Chabot indicates that the rate of silting even varies between different periods of time in the same reservoir. For determining the rate of erosion from a drainage area, the quantity of soil removed annually expressed in cubic feet per acre has been found useful, although the silt deposit expressed as a percentage of the total runoff is also of value. An examination of Table 4 will show that the rate of erosion varies from less than 1 cu.ft. per acre annually up to a maximum of 387 cu.ft. per acre annually. An erosion of 550 cu.ft. per acre per year has been reported by the U. S. Soil Conservation Service, but the details are not available.

The erosion from the watershed is governed by many factors which include the total annual rainfall, the maximum intensity of rainfall, the runoff (both the total annual runoff and the maximum rates of runoff), the slope of the watershed, the character of the soil, the character of the bedrock, the character and extent of vegetation both along the stream channel and upon the entire watershed. The silting of a reservoir is also affected to a marked degree by the total capacity of the reservoir in relation to the mean annual runoff of the stream. When this ratio is low, it also can be controlled to a greater extent by passing of silt-laden water though sluice gates at the base of the dam or over the spillway. The construction of upstream reservoirs may greatly alter the rate of silting in the given reservoir, and the proper control of the watershed lands can be effective in reducing silting to a minimum. The effects of rainfall and runoff on the rate of erosion can be seen by a comparison of the silt deposits in the various District reservoirs. In Chabot Reservoir

TABLE 4

Measured Rates of Silt Deposition in Reservoirs

RESERVOIR	STREAM	DRAINAGE AREA IN SQ.MI.	MEAN ANNUAL INFLOW IN 1000 ACRE-FT.	SILT DEPOSIT TOTAL ACRE-FT.	PERIOD OF YEARS	SILT DEPOSITED		
						Annually Acre-Ft.	Per cent of Run-off	Annual Cu. Ft. per Acre
Lake Chabot	San Leandro, Calif.	42	18.8	3,300	48	69	0.37	112
Upper San Leandro	San Leandro, Calif.	30.3	8.2	170	11	15.4	0.19	34.5
San Pablo	San Pablo, Calif.	32.2	8.8	1,133	21	54	0.61	114
Gibralter	Santa Ynez, Calif.	200	16.4	4,100†	14	293	1.79	99.7
LaGrange	Tuolumne, Calif.	1,500	1,863	1,260	10	126	0.0068	5.71
Sweetwater	Sweetwater, Calif.	181	18.9	5,760	39	148	0.78	55.5
Roosevelt	Salt, Ariz.	5,760	840*	108,800	25	4,352	0.52	51.4
San Carlos	Gila, Ariz.	13,540	305	36,896	6.33	5,829	1.91	29.3
Elephant Butte	Rio Grande, N. Mex.	26,312	1,091	365,186	20.25	18,034	1.65	46.6
Zuni	Zuni, N. Mex.	650	23.9	11,940	20	597	2.50	62.4
McMillan	Pecos, N. Mex.	22,000		41,400‡	21.42	1,933		5.97
McMillan	Pecos, N. Mex.	22,000		5,000‡	17	294		0.91
Old Lake Austin	Colorado, Texas	38,200	2,370	23,600	6.75	3,500	0.15	6.23
New Lake Austin.	Colorado, Texas	38,200	1,970	30,600	13	2,350	0.12	4.18
Lake Penick	Clear Fk. Brazos, Tex.	2,250	111*	965	7	138	0.12	6.15
White Rock	White Rock Ck., Tex.	114		3,882	25	155		92.5
Rogers	Little, Texas	0.55		37.5	12	3.1		387
Lake Waco	Bosque, Texas	1,662		4,766	5	953		39.0
Lake Worth	W. Fk. Trinity, Tex.	1,870	212*	13,900	13	1,060	0.50	38.7
Crook	Pine, Tex.	51.6		732	13.1	55.9		108
Gibbons	Pine, Tex.	1.26		78	36	2.2		172
Lake Spavinaw	Spavinaw Ck., Okla.	400		1,177	11	107		18.2
Lake Sapulpa	Euchee Ck., Okla.	8.72		183	22.5	8.1		93.3
Guthrie	Cottonwood Ck., Okla.	13.3		456	14.5	31.5		161
Boomer	Boomer Ck., Okla.	9.13		171	10.25	16.7		124
Lake Booneville.	Petit Jean Ck., Ark.	2.6	2.22*	7.88	6.74	1.17	0.053	30.6
Lake Eldorado	Walnut, Kans.	33.0		131	9	14.6		30.5
Lake Olathe	Cedar Ck., Kans.	6.2		55	4.9	11.2		125
Ottawa Lake	Sand Ck., Kans.	20.5		71	8	8.9		29.5

* Average Runoff, but not actual inflow during silt measurements.

† Rate of silting has been increasing, following brush fires on watershed.

‡ First survey for period 1893 to 1915. Second survey 1915 to 1932. Heavy growth of Tamerisk or Salt Cedar developed along stream after 1915.

TABLE 4—*Concluded*

RESERVOIR	STREAM	DRAINAGE AREA IN SQ.MI.	MEAN ANNUAL INFLOW IN 1000 ACRE-FT.	SILT DEPOSIT TOTAL ACRE-FT.	PERIOD OF YEARS	SILT DEPOSITED		
						Annually Acre-Ft.	Per cent of Run-off	Annual Cu. Ft. per Acre
Mission Lake	Mission Ck., Kans.	11.4		289	13	22.2		136
Wellfleet	Medicine Ck., Nebr.	43§		55	5.6	9.8		44.9
Lake Taneycomo	White, Mo.	4,610		20,266	22.4	905		13.6
Baker	Sandstone Ck., Mont.	5.2		254	29.1	8.7		119
Black Canyon	Payette, Idaho	2,540	2,235	4,037	12	336	0.015	9.0
Guernsey	North Platte, Wyo.	16,200	1,670	8,400	5.9	1,430	0.085	8.85
Boysen	Bighorn, Wyo.	7,740	1,500*	13,000	13	1,000	0.067	87.7

§ Only 15 sq.mi. with surface drainage.

SOURCE OF DATA: Page 210, ref. 2; ref. 1; Advance Reports Soil Conservation Service, U. S. Dept. of Agric.; and Records of East Bay Municipal Utility District.

the rate of silting varies from a minimum of 61 cu.ft. per acre, or 0.23 per cent of the total runoff, during a period that the mean annual runoff averaged 392 acre-ft. per sq.mi., to a maximum of 212 cu.ft. per acre, or 0.59 per cent of the total runoff, during a period that the mean annual runoff averaged 526 acre-ft. per sq.mi. The average rate of erosion for the entire 48-year period was 112 cu.ft. per acre, or 0.37 per cent of the total runoff, with an average annual runoff of 447 acre-ft. per sq.mi.

In the survey made on Upper San Leandro Reservoir an erosion of 34.5 cu.ft. per acre, or 0.19 per cent of the total runoff, was measured during a period when the runoff averaged 271 acre-ft. per sq.mi., while on the San Pablo Reservoir an annual erosion of 114 cu.ft. per acre, or 0.61 per cent of the total runoff, was measured during a period when the mean annual runoff averaged 274 acre-ft. per sq.mi. If it is assumed that proportioning the rate of silting of San Pablo reservoir to that in Lake Chabot, according to the observed relative runoff per square mile of the respective drainage areas, will result in a true estimate of the probable rate of silting in San Pablo Reservoir over a period of time having a higher rate of runoff; an erosion of 185 cu.ft. per acre annually will be determined ($114 \times \frac{447}{274} = 185$).

With normal runoff which would have yielded an average of 521 acre-ft. per sq.mi. from the Upper San Leandro watershed and 422 acre-ft. per sq.mi. on the San Pablo watershed during the same period of 48 years that silt deposits were measured on Chabot Reservoir, there is no reason why an erosion as great as or greater than that observed on the Chabot Drainage area would not occur on these other watersheds. The formations and slopes are similar on both areas and, in fact, the entire area above Upper San Leandro Reservoir is included in the drainage area formerly tributary to Chabot Reservoir. The extent of heavily wooded areas on the San Pablo and Pinole drainage areas is definitely less than on the San Leandro watershed so that one might expect a greater rate of erosion over a long period of time on these two drainage areas over that which would occur on the San Leandro watershed. The rate of erosion on the District's watershed lands is one of the highest rates in the western United States, and about two to three times greater than that observed on the such notoriously silt-laden streams as at Salt Reservoir at Roosevelt Dam and the Rio Grande at Elephant Butte Dam. Of course, on these two streams, due to the lower rate of runoff, the total amount of silt expressed as the percentage of the runoff is higher than on the District's watersheds.

The effect of vegetation in controlling erosion can be cited in connection with results obtained at the MacMillan Reservoir located on the Pecos River in New Mexico. During the period from 1893 to 1915 the average rate of silt deposits in the reservoir indicated a rate of erosion of 5.97 cu.ft. per acre of drainage area annually. However, during the period from 1915 to 1932, the rate of erosion was reduced to 0.91 cu.ft. per acre annually. This remarkable reduction in the erosion resulted from the growth of Tamarisk or Salt Cedar, which was accidentally introduced into the stream channel about 1912. By 1915 the growth was well developed and ultimately spread for a distance of 200 mi. upstream from the reservoir, with the very pronounced reduction in the rate of silt deposits cited above (page 15, ref. 1). The opposite effect can be observed in the rate of silt deposits in the Gibralter Reservoir, located on the Santa Ynez River and used as a source of municipal water supply for the City of Santa Barbara. Subsequent to 1920, when the reservoir was constructed, a series of fires burned off the vegetation in various years so that by 1933, 83 per cent of the drainage area had been burnt off. The topography of the drainage area is very mountainous

and the sedimentary formations throughout the area have broken down into deep and friable soils, which eroded rapidly after the protective cover of vegetation had been removed. As a consequence the rate of erosion increased from 54 cu.ft. per acre annually between 1920 and 1925 to 170 cu.ft. per acre between 1925 and 1931 and to 204 cu.ft. per acre between 1931 and 1934 (page 20, ref. 1).

The rate of erosion is naturally affected by the character of the soils and underlying bedrock formations. For example, at the La Grange Reservoir on the Tuolumne River in California the average rate of erosion, as measured by the silting of the La Grange Reservoir, was 5.71 cu.ft. per acre annually during the first ten years that the reservoir was in operation. In subsequent years the rate of silt deposits in this reservoir decreased due to the partial filling of the relatively small pond above the dam and the consequent passage of a large quantity of silty water over the spillway during the high flood flows (page 211, ref. 2). Similarly at the Black Canyon Dam on the Payette River in Idaho, the rate of erosion has been 9.0 cu.ft. per acre annually (Advance Reports of U. S. Soil Conservation Service). On both of these drainage areas the formation is largely granitic. On the other hand, the rate of silting of the Roger Municipal Reservoir, located on the Little River in Texas, has been 387 cu.ft. per acre annually. The drainage area above this reservoir is located in the black land soils of Texas, a formation which is notably erosionable under the current agricultural practices of that locality (page 57, ref. 1).

If it is possible to open sluice valves in the bottom of the reservoir and to discharge a portion of the water following the most turbid runoff periods, it will be found that the silt-laden water will settle to the bottom of the reservoir and flow along the reservoir bottom so that a large percentage of it can be discharged directly through the sluice valves. This discharge must be made before the fine silts, which are held in suspension, have had time to settle out. By this method of control much of the fine material which forms the bottom-set beds can be disposed of and sedimentation largely limited to delta deposits at the head of the reservoir (page 720, ref. 3). This method of control has been practiced at the Pardee Reservoir.

The water of the Mokelumne River is not of high turbidity at any time and during much of the year it is very clear. The maximum turbidity rarely exceeds 250 p.p.m. With the valves open, during periods of turbid inflow, water of approximately the same turbidity

can be observed discharging through the sluice valves. This is particularly noticeable during the snow melt period when incoming and outgoing water frequently have a characteristic milky color found only in water from the high altitudes. On one occasion in June, 1935, when the sluice valves were closed during the high flow from melting snow, turbid water was observed to rise and pass over the spillway. By opening the valves to reduce the spillway discharge to a small quantity, the release of this turbid water was transferred to the sluice valves. The river discharge at that time varied between 3,000 and 5,000 sec.ft., and the turbidity from 20 to 30 p.p.m.

Life of District Reservoirs

Due to the many factors influencing the rate of erosion from a watershed, it is difficult to project actual observations into the future with the view of estimating the probable life of any reservoir. With this reservation in mind, it is concluded, as a result of silting observations which have been made on the District's reservoirs, that an average rate of silting of 112 cu.ft. per acre annually is reasonable over a long period of time on the San Leandro watershed, provided the conditions controlling erosion remain in the future substantially as they have been in the past, whereas 185 cu.ft. per acre annually would be representative under similar assumptions of the average erosion which now exists on the San Pablo, Lafayette, and Pinole Watersheds. Using these figures, the life of the various District reservoirs has been determined as shown in Table 5.

Within a period of 100 years, 18,900 acre-ft. of storage would be lost on the existing local reservoirs of the District, provided the erosion continued at its present rate. Since all of the available storage would be required at the end of the hundred-year period in order to regulate properly and distribute the District's water supply, it is obvious that the storage which had been lost by silting would have to be replaced either by the construction of new reservoirs or by the raising of the present dams and the increasing of the capacity of the existing reservoirs. In the Pardee Reservoir, since the results of the silt survey have not yet been completed, the probable rate of silt deposits has been estimated by comparison with the Tuolumne River in California and the Payette River in Idaho where runoff and soil conditions are similar. On these two streams the average erosion of 7.4 cu.ft. per acre annually has been observed. Frank Bonner has determined as the result of measurements that the total

silt load of the Tuolumne River averages 15.8 cu.ft. per acre annually (page 309, ref. 4). The annual silting of the Pardee Reservoir, therefore, will probably be between a minimum of 42 and a maximum of 90 acre-ft. per year over the 387 sq.mi. from which watershed erosion directly affects this reservoir. With the reservoir operation as practiced, namely, opening the sluice valves in the dam to discharge a part of the silt-laden waters, the annual rate of silting should tend toward the lower value. Hence, the life of this reservoir should be nearer to the maximum of 5,000 years than to the minimum of 2,360 years.

Control of Erosion

During the past decade methods of controlling erosion have come to the foreground. This has largely been brought into prominence

TABLE 5

Lives of the District's Reservoirs

RESERVOIR	RATE OF SILTING ACRE-FT. PER YEAR	EXPECTED LIFE IN YEARS
Chabot	19.4	650
Upper San Leandro	50.0	830
San Pablo	87.7	490
Lafayette	3.6	1,240
Pinole	28.5	1,160

due to the gradual removal of formerly fertile agricultural lands from cultivation as the result of the removal of the top soil by erosion following improper agricultural or grazing practices. The methods of controlling erosion which have been developed in connection with soil conservation can be used to equal advantage in the control of the silting of reservoirs. In soil conservation, engineering and forestry must necessarily work together. In order to control the rate of erosion from any given drainage within the limits of its geologic norms, existing vegetation on both wooded and grassland areas must be cared for and fire prevented or controlled by adequate patrol. New plantings should be set out on all areas denuded by forest fires. Where cultivation exists, proper agricultural practices on side hill fields must be observed and over-grazing of pasture lands prevented.

In addition to this work in controlling the vegetation growth through the watershed area, check dams must also be constructed

along the steep drainage channels. These check dams are not constructed primarily to catch silt, although that function naturally follows, but their primary purpose is to stabilize the stream channel in order to minimize the rate of erosion and to permit reforestation to go forward. Before embarking upon any program of watershed control, a survey of the area should be made to determine the various controlling factors: first, all points of erosion should be noted and distinguished between sheet erosion, gulley formation, and slides. Also the land use should be classified between wooded areas, grazing land, cultivated land, and suburban real estate developments. The general slope of the land over various portions of the drainage area should also be noted and the soil texture distinguished between sandy loam, silt loam, loam, clay loam, or clay. The soil profile should also be established to distinguish between residual soils, resting on sedimentary rock or on basic igneous rock; and alluvial soils, either ancient mature soils (consisting mainly of hard pan or clay pan) or more recent unconsolidated alluvium.

To provide for the control of mountain streams, a system of check dams, together with debris dams, must be carefully planned. The profile, alignment, and cross sections of the various stream channels must be made available for a preliminary design. The dams may be either overflow dams of concrete or rubble masonry or earth dams provided with adequate spillways. Of the latter, a dam known as the "Missouri" type is frequently used. The spillway in this dam consists of a pipe outlet provided with a vertical drop inlet. Many detailed descriptions of the various types of control have been published in the engineering literature during recent years (see page 355, ref. 5; page 675, ref. 6; ref. 7; pages 393–6, ref. 8).

The purpose of the check dams is to stabilize the streambed and the side slopes of the channel so as to hold the soil in place and to permit the vegetation to grow. The primary object is not to impound silt, since if this were the only function of these dams, it could be readily demonstrated that the construction of additional reservoir storage would be more economical than the small debris dams, due to lower unit cost per acre-foot of storage capacity of the large storage reservoirs.

The alleged function of the check dams in stabilizing the side slopes may appear overly optimistic, but it becomes rational when the process of erosion is quite clear. Erosion in the channel usually accompanies heavy rains and the resulting floods. The eroded

stream channel is the result of the drag of the water on the bed, which tends continually to deepen the channel and make the side slopes steeper. This drag, in accordance with du Boy's theory, is proportional to the depth of water and to the slope or gradient of the channel. The erosion of the stream bed, which naturally follows flood runoff on steep mountain water courses, removes the support from the banks, which are usually saturated at the time of maximum erosion and in their most unstable condition. Hence, they readily slip into the channel and the material is carried away by the flowing water. This process is gradually repeated and extended up the smaller side channels, with the result that the fertile top soil is carried away eventually by the stream. With the introduction of the check dams, deposits of silt behind these structures tend to support the side banks and to prevent the sloughing. A system of check dams also reduces the gradient of the stream channel; and by building up the bed, widens the channel and consequently reduces the depth of the water. Both of these factors; namely, lessened depth and flatter gradient, reduce the bottom tractive force or drag of the flowing water and consequently reduce the erosion in the main channel. The stability of the stream bed throughout the drainage system is improved and vegetation along the channel can flourish so that erosion over the watershed is progressively decreased.

Erosion control of this type has just been started on the local watersheds of the District, with the cooperation of the Soil Conservation Service of the U. S. Department of Agriculture and the Works Progress Administration. No statement can be made at the present time of the effectiveness of this proposed control, but the situation is very favorable for a definite evaluation of the effectiveness of this type of erosion control. The long records of rainfall, runoff and other hydrologic data, coupled with the 48-year measurements of silting, indicating a high rate of erosion on the San Leandro and San Pablo watershed, present an opportunity for a detailed study of the control of reservoir silting not duplicated in many localities.

It is difficult to estimate the future worth of the District's local reservoirs. That many additional reservoirs will be needed in the future in excess of the existing storage is admitted. The development of new reservoir sites to replace those which had silted will become increasingly costly. Every highway, every railroad, the grazing of livestock and the cultivation of fields, in other words, every

man-made improvement which results in a scar on the earth's surface introduces an incipient source of erosion that may reach serious proportions. This is particularly true on easily erodible materials such as found on the District's local watershed. If the life of the storage reservoirs can be extended at a reasonable cost by soil conservation methods, it is well worth the effort.

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Mono Craters Tunnel Construction Problems

By H. L. Jacques

THE Mono Basin Project is being constructed in order to increase the water supply of the city of Los Angeles by approximately 145 second feet by extending the Los Angeles Owens River Aqueduct into Mono Basin, thereby bringing the aqueduct to its full carrying capacity of 480 sec.-ft. It is financed by a bond issue, which was voted by the citizens of Los Angeles in 1930, for the purpose of making general improvements and extensions of the City's water supply system. It is situated in Mono County approximately 350 miles due north of Los Angeles. The source of water is a group streams which drain the east slope of the Sierra into Mono Lake. Named from the north, the streams intercepted are Mill, Leevining, Walker, Parker, South and Rush Creeks.

The project comprises the construction of two large impounding and regulating reservoirs, one on Rush creek in Mono Basin with a capacity of 48,000 acre feet, the other at the head of the Owens River Gorge in Long Valley with a capacity of 163,000 acre-ft.; 15.42 miles of conduit; and approximately 67,000 ft. of tunnel.

The main feature of the project is the construction of the Mono Craters Tunnel, 59,812 ft. in length, through the mesa separating the Mono Basin and the Owens River watershed. Mono Basin is 662 sq.mi. in extent with Mono Lake (at an elevation of 6,419 ft. and covering an area of 54,720 acres) occupying its lowest part. The lake is fed mainly from the streams on the east slope of the Sierra and partly by springs and underground sources from the lava covered mesa to the south and east. At an earlier period the lake was several hundred feet higher in elevation than at present.

The eastern slope of the Sierra in this region is characterized by

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A paper presented on October 26, 1939, at the California Section Meeting at San Francisco, by H. L. Jacques, Engineer of Major Construction, Bureau of Water Works and Supply, Department of Water and Power, Los Angeles.

the presence of great glacial moraines extending from the base of the mountains far into the valley below.

To the south the boundary of the basin is partially formed by a series of eruptive volcanic cones, known as the Mono Craters, from which the tunnel derives its name. Some twenty in number, these volcanic peaks, attaining an elevation of 9,000 ft. or more, extend in a north and south direction for about 10 miles. Most of them are of very recent origin, being formed subsequently to the glacial flows that cover the ancient floor of the valley. For the most part, they are composed of acidic lava, ranging from fairly compact obsidian, a black volcanic glass, to light grey pumice. For a distance of approximately 10,000 ft., the tunnel penetrates these volcanic cones.

The mesa separating the basin and the Owens River along the line of the tunnel for about 8 miles has an elevation of approximately 8,000 ft.

The mesa occupies part of what was once a deep valley lying between the Sierra on the west and the White and Inyo Mountains to the east, each range rising to an elevation of 13,000 ft. or more. Portions of the floor of this ancient valley were covered with lakes which, at a later period, were filled with products of erosion from the steep slopes of the surrounding mountains.

During the quaternary period great glaciers, of which remnants are still to be found, existed in the High Sierra forming the west and southwest boundary of the Basin. In time, these ice and snow fields with their included debris built great moraines at the base of the mountains and far out onto the valley floor. At a later period, the region was visited by great volcanic activity resulting in the formation of high volcanic peaks, of which the Mono Craters are a part and the covering of the surrounding terrain for miles with lava and volcanic ash.

The Mono Craters Tunnel from portal to portal is driven through such formations.

Beginning of the Project

Preliminary surveys for the project were started in 1930. Several possible routes were studied. Borings by diamond and calyx drills were made on two other routes besides on the one adopted.

For the most part, the surface of the plateau separating the two watersheds is covered with volcanic ash of variable depth so that

very few outcrops of the underlying rock formations are visible, making geologic determinations from surface indications practically impossible. Eight drill holes varying in depth from 386 to 1,003.5 ft. and from 5,360 ft. to 10,130 ft. apart were bored along what appeared to be the most favorable line for the tunnel.

The percentage of cores obtained from several of the holes at their lower depths was so small that it was difficult for even competent geologists to classify properly the true character of the formations at or near tunnel grade. One of the consulting geologists in his report recommending the location adopted, stated that, while great difficulties were to be anticipated in driving the tunnel, he believed none would be found to be insurmountable. He was right.

Preliminary construction, comprising roads, telephone and power lines, the building of water supply and the establishment of camps, was started in July, 1934. Excavation was started at West Portal in September and at East Portal in November, 1934.

West Portal, at an elevation of 7,058 ft., is situated 7.8 miles southeast from the town of Leevining. From this point, the tunnel runs south $55\frac{2}{3}$ degrees east for 42,825 ft. to an angle point; thence south $43\frac{1}{3}$ degrees east for 16,987 ft. to East Portal on the Owens River—a total length of 59,812 ft.

The tunnel has a gradient of .0005, is almost circular in cross-section with a diameter of 9 ft. $7\frac{1}{2}$ in. inside the concrete lining. Based on a coefficient of $n = 0.012$, it will have a carrying capacity of 365 sec.-ft. Excavation was completed April 26 this year when Headings 3 and 4, driven from Shafts 1 and 2, respectively, met on that date. The concrete lining is 96 per cent completed, 2,708 ft. remaining to be lined October 24, 1939.

With the completion of the three miles of conduit, now under construction, connecting the East Portal of the 3,450-foot Grant Lake Reservoir outlet tunnel with West Portal, Mono Craters Tunnel, it is expected that the waters of Rush Creek amounting to a mean annual flow of 51 sec.-ft., plus some 30 sec.-ft. made in the tunnel itself, will be delivered into Owens River early next year.

As shown in the accompanying profile, excavation was carried on from six headings; first from Headings 1 and 6, West Portal and East Portal, respectively; then from Headings 4 and 5 from Shaft 2 and from Headings 2 and 3 from Shaft 1. Each shaft has three compartments, two for hoisting, 5 ft. by 5 ft. 2 in., and a manway and utility compartment $5 \times 6\frac{1}{2}$ ft.

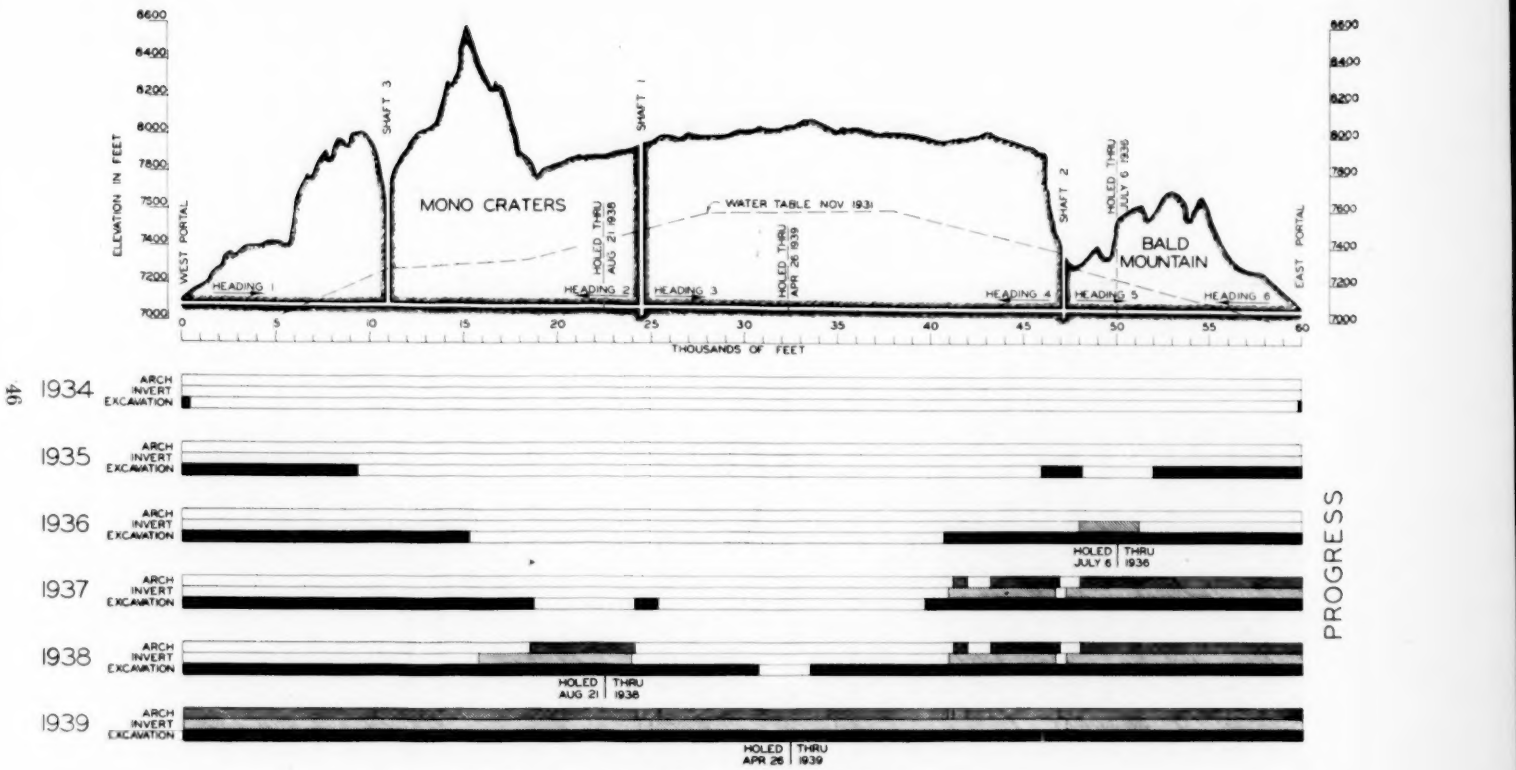

FIG. 1. Mono Craters Tunnel Profile

Shaft Sinking Problems

Shaft 2, 12,735 ft. from East Portal, is 299 ft. deep from collar to tunnel invert and 361 ft. to bottom to allow for skip loading below muck pocket. This shaft presented no difficulties in sinking, which was started on February 9, 1935 and completed to tunnel grade four months later.

Formations encountered from the surface were: benching for the collar was in hard rhyolite tuff; from the collar 7 ft. of loosely cemented volcanic ash, 207 ft. of streambed clayey gravel, 113 ft. of basalt; and from 28 ft. below tunnel invert to bottom of shaft, a fairly well cemented glacial gravel.

The shaft being located on the rib of a small spur jutting out from high cliffs surrounding a small green meadow fed by springs, indicated considerable water might be encountered. Consequently, a 16-inch well was drilled about 10 ft. from one side of the shaft at the time sinking was started. After penetrating about 20 ft. into the hard basalt, with no increase of water, the hole was bottomed at 234 ft. from the collar. A deep well pump kept the shaft thoroughly dewatered as sinking progressed. The well made approximately 150 gal. per min. which undoubtedly materially facilitated sinking.

The hard basalt was excellent formation in which to excavate a spacious muck pocket and a large pump sump on each side of the shaft, also for the enlargement of the tunnel section for a battery charging station and the construction of a repair drift opposite the shaft. The shaft is lined with concrete for 40 ft. from the collar down. The shafts are constructed 20 ft. from the center line of the tunnel.

Driving was begun in Heading 4 August 28, and in Heading 5 September 7, 1935.

The construction of Shaft 1, 24,460 ft. from West Portal and 22,614 ft. from Shaft 2, presented a different problem. Sinking was started December 3, 1934, and tunnel grade, 896 ft. from collar, was reached May 12, 1937. The bottom is 48 ft. deeper to take care of the muck. From the collar, the formations are as follows: 18 ft. loose volcanic ash; 507 ft. fairly hard rhyolite tuff; 38 ft. volcanic ash; 308 ft. of glacial gravels; 73 ft. sandstone (old rock floor).

Sinking through the upper 525 ft. was made in good time. Water was encountered at a depth of 492 ft., or 33 ft. above the bottom of the hard rhyolite. When the softer formation, the ash, was encountered, trouble began and continued on down to tunnel grade.

A few feet of sinking below the water table brought water in large quantities, 200 gal. per min. or more at the start. A chamber was cut in the hard rock 480 ft. from the collar for the installation of pumps for boosting the water to the surface. The volcanic ash, in all degrees of fineness, was hard to hold as it would seep through the lagging or spiling or boil up from the floor, causing much cavitating back of the timbers. Below the ash the glacial gravels and other fragmental materials were even more troublesome.

Glacial gravels, as found at the base of the Sierra in the form of moraines, are a heterogeneous mass of debris ranging from boulders many cubic yards in volume down to exceedingly fine rock meal. In its initial state the whole mass is fairly well cemented and generally impervious to water. This is exemplified in the numerous lakes found on the eastern slope of the Sierra formed by terminal moraines across canyons and water courses. Grant Lake on Rush Creek is an example, and the new Grant Lake Dam-site has its abutments in a terminal moraine through which the stream had ultimately cut through.

In the periodic advance and recession of the ice field, the moraines were subjected to periods of erosion and sedimentation in which the materials were segregated and classified, resulting in beds of open coarse gravel, coarse sands, fine sands and silts.

While large boulders, of such size as to require blasting, were often encountered in the shaft, the greatest trouble was found in the fine sands and silts which, due to the water pressures behind or under them, as they were approached, would cause them to boil up and fill the shaft for many feet. At several places, as aquifers, or water bearing strata were encountered, small drifts were constructed leading from or around the shaft to intercept and control the water.

When construction of this shaft was started, some consideration was given to the desirability of sinking a well or drill hole as had been done at Shaft 2 and of installing a deep well pump to facilitate drainage. Calyx drill hole No. 4, 6,155 ft. to the west, and drill hole No. 5, 3,388 ft. to the east on the tunnel line, indicated the water table to be at an approximate elevation of 7,449 ft. at the shaft, or 492 ft. below the collar. That assumption proved to be correct. It was considered an unnecessary expense to drill such a well when sinking for half the depth or more would be in dry ground. Later experience draws the conclusion that the well would have been most desirable and economical.

Ultimately a 12-inch well, drilled to tunnel grade from the then

bottom of the shaft 131 ft. above tunnel grade, helped tremendously in sinking operations. While no pumping was done from the well, it intercepted water bearing strata alternating with less pervious fine sands and clayey deposits, relieving the latter from uplift from water pressure as they were approached. The shaft is concrete lined from the collar down for 40 ft. From 40 to 520 ft., or to a level 5 ft. above bottom of the section in hard rock, it is gunited. Below that point, the shaft is lined with reinforced concrete, and octagonal in shape, better to withstand the heavy ground and water pressures. After concreting, the walls were backed with cement grout under considerable pressure. Holes were drilled later through the walls for drainage.

At a place 709 ft. from the collar, 250 gal. per min. of water was intercepted which, for a year or more, was used for camp domestic supply with no material decrease in volume. During period of sinking, the average flow in the shaft was approximately 550 gal. per min. and at times considerably higher.

Driving the Headings

The formation encountered in advancing Heading 1 for 17,300 ft. was mostly rhyolite which, as a whole, was fairly hard and required occasionally only light support. For the first mile from West Portal, small spurs of granite were penetrated with intervening valleys refilled with glacial gravels, volcanic ash and detrital materials. (See Geological profile.)

Water was encountered at 6,100 ft. At 7,800 ft. carbon dioxide gas was detected in small quantities. As progress advanced, both water and gas increased materially. In May, 1936, at 13,000 ft. from the portal the gas had increased to such a volume that the ventilating facilities were no longer adequate, and it became necessary to suspend driving at the face. At 11,000 ft. from the portal a ventilation shaft, known as Shaft 3, was constructed by sinking and raising. A two-compartment shaft, 535 ft. deep, it was completed in 38 days.

At the shaft collar were installed several blowers with a total capacity of over 50,000 cu.ft. per min. Electrically operated, each unit had an auxiliary gas engine attached to operate the blower in case of a power outage, assuring adequate ventilation at all times. Advance at the face was resumed in July. At approximately 14,400 ft., the neck of one of the main craters was crossed which caused considerable trouble. For several hundred feet, the formation was badly broken indicating a major fissure or fault, probably one through

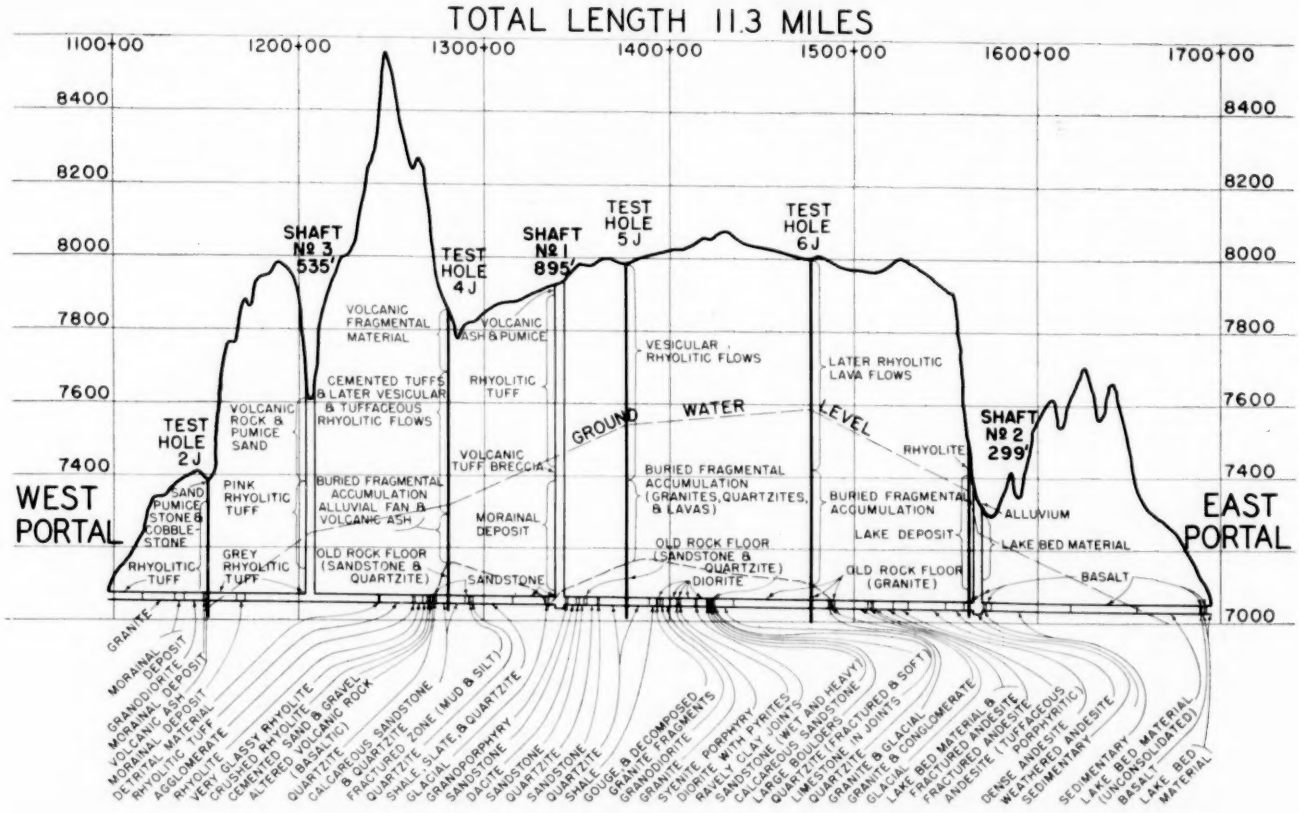

FIG. 2. Geological Profile, Mono Craters Tunnel

which the craters erupted. Large quantities of water and some warm springs (up to 97°F.) highly charged with carbon dioxide, were intercepted. At one place, high water pressure back of the face caused a run-in, filling the tunnel for a hundred feet.

On reaching 15,230 ft., some large crevices were intercepted which increased the water flow by several thousand gallons per minute, temporarily flooding the tunnel. Work in the face was suspended to construct additional pumping installations, one at Shaft 3 and one at Station 15,000, each of 9,000 gal. per min. capacity.

On resuming work in the face, grouting off the water and gas was resorted to. The procedure consisted of drilling deep holes in the face, from four to eight in number up to 40 ft. in depth, then to pump cement grout in them to refusal at 1,000 to 1,100 lb. per sq.in. pressure. By using quick setting cement to finish the grouting of each hole, drilling and blasting at the face followed immediately after finishing grouting.

After grouting, the face was advanced 20 ft. and again followed by the drilling of the 40-foot holes. If water and gas in troublesome quantities were not encountered, the face was advanced another 20 ft., otherwise the face was again grouted. This procedure assured at least a 20-foot grouted wall ahead of the face at all times, decreasing the risk of run-ins on approaching soft or broken ground.

To reduce further the dangers of cave-ins on approaching soft zones, the water pressures were relieved by drilling holes in the walls and roof back of the face. Pipes placed in the drains would convey the water and gas back to pumps, thereby reducing the amount of water on the tunnel floor and keeping the air more free of carbon dioxide gas.

At 17,300 ft., Heading 1 passed through a transition zone of agglomerates consisting of debris from volcanic rocks and underlying alluvial deposits, and entered into the ancient rock floor, old metamorphics consisting of quartzites, sandstones, shales and slates. For the most part, these old sedimentaries are thinly bedded, have been subjected to great disturbances resulting in much distortion, fracturing and weathering. The bedding planes are usually found dipping at a steep angle between the vertical and 60 or 70 degrees, commonly toward the north. The strike is generally in an east and west direction. Their vertical attitude permits free percolation of water from the overlying alluvial and glacial gravels and other pervious materials. For about 3,000 ft. after entering the sedimentaries, the sandstone is calcareous and carbonates were generally

found in the joints of the quartzites and shales, a condition which probably accounts for the presence of carbon dioxide gas.

Fragments of granite and limestone were found in driving through the main vent of the crater, indicating also that the eruption must have penetrated upward through old granites and limestones.

The source of carbon dioxide is most likely in these calcareous formations which have been broken down by volcanism and the action of sulfurous and other acids commonly found in active volcanoes. The gaseous zone extends approximately to 19,500 ft. from the portal or for a total distance of 11,700 ft. No carbon dioxide was found beyond that point anywhere in the tunnel. Since reaching the center of the craters the volume of gas up to the beginning of this year has averaged about 1,000 cu.ft. per min., the maximum approaching 1,400 cu.ft. per min. About 40 per cent of the carbon dioxide is generally retained in the water, the remainder being freed in the air. The volume of gas liberated depends considerably on the condition of the atmosphere, a great increase occurring during a rapidly falling barometer with a corresponding decrease of carbon dioxide with a rising barometer.

During periods of heavy gas, daily records were kept of the condition of the air flowing through the tunnel, the volume checked and the air analyzed for carbon dioxide and oxygen contents. It was endeavored to keep the percentage of carbon dioxide below 2.5 and the oxygen above 20.5. While carbon dioxide is neither toxic nor inflammable, an excess of it reduces the oxygen content of the air, making respiration laborious and exhausting.

On some occasions, while advancing the face, when large flows of water were encountered, gas was liberated so quickly and in such volume that the air would become saturated to 15 or 20 per cent, although the air line might be delivering some 8,000 cu.ft. per min. of fresh air within a few feet of the face. At such times the miners had to leave the face and seek a safer place, regardless of whether or not a run-in from heavy ground, water or gas pressure was impending.

The ground within the metamorphic zone was very heavy, requiring heavy support throughout to the holing-through point with Heading 2 at 22,426 ft. from the portal, and beyond throughout Headings 2 and 3 and into Heading 4 where the metamorphics come in contact with granite. It is most variable in character, changing

from hard to soft or vice versa within a few feet. The tunnel is driven for 3½ miles through this formation. The area has been broken up by numerous faults running more or less in a northerly direction. Fissures filled with loose sand, silts, and clays were frequently encountered, which, generally accompanied with water, were very difficult to penetrate. The bulk of this area was heavily spiled and much of the distance was advanced behind breast boards.

Control of Water

Shortly before holing through Headings 1 and 2, Heading 1 was making 9,300 gal. per min. of water, about half being pumped to West Portal, the remainder up Shaft 3.

Due to the flat gradient and small width of tunnel, the water had to be picked up at close intervals. Pick-up pumps were quite uniformly spaced 1,000 ft. apart as were transformer stations. At places where carbon dioxide entered in a large volume it was necessary to place the pumps at much closer intervals to prevent the escape of gas from the water into the air. Carbon dioxide is especially corrosive to steel. Pick-up pumps pumping carbonated water were made of bronze and had bronze rotating elements. Any other material did not last long.

Main pumping stations, each of 9,000 to 10,000 gal. per min. capacity were installed at 7,800 ft., 11,000 ft., 13,500 ft. and 15,000 ft. from the portal, each generally made up of two units. Dewatering lines were two or three 14-inch #12 gage steel pipe flanged in 24-foot sections well anchored to the wall or floor.

Method of Support

At start of construction, timber sets made up of 8 by 8-inch timber were used for a few hundred feet at each portal heading; later steel sets of 5-inch 12¼-pound I-beam and 6-inch 12½-pound I-beams were used in lighter ground. For the last 25,000 ft. of tunnel driven, for the most part in heavy ground, 6-inch H-beams of 20- and 30-pound weight per foot were used throughout. In the heaviest ground, where 30-pound beams were used, a steel spreader was also employed made up of 6-inch 20-pound H-beams curved to a radius of 19 ft. 4 in. The sets are normally placed on 5-foot centers, though in many places they are much closer and in numerous cases only a few inches apart. While in ground requiring but normal timbering,

the sets are generally lagged with 2x6 in. or 3x6 in. lumber, or crown bars of 4x6 in. or 6x6 in. timber, in heavy ground, spiling made up of 4x6 in. or 6x6 in. timbers had to be driven well beyond the position of the following set to prevent runs in soft material. In wet ground containing boulders and inclined to run, steel spiling made up of 6x2 in. 10.5-pound steel channels was used advantageously. Spiling was generally driven by the mucking machine acting as a battering ram, a long timber with reinforced ends being placed between the mucker and the spiling driven.

In driving Heading 2, westerly from Shaft 1, the contact between the old rock floor and the overlying gravels was followed for a con-

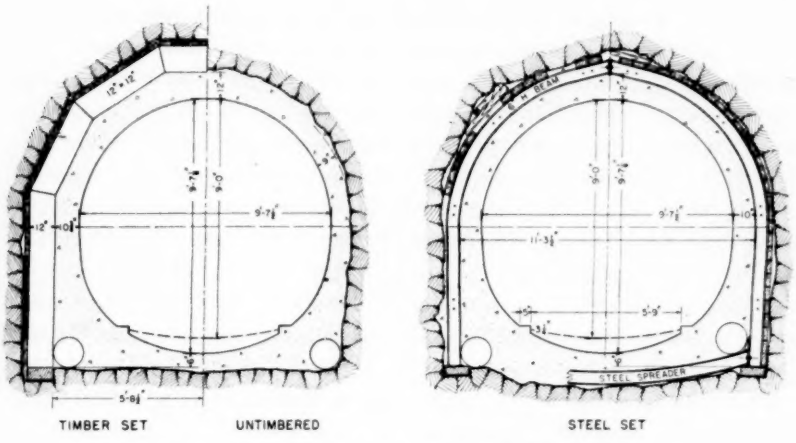

FIG. 3. Cross Sections, Mono Craters Tunnel

siderable distance, going in and out of the sandstone and into the gravels. At every change considerable additional water was encountered, which with the highly weathered condition of the sandstone made for a tendency to run unless extreme care was taken in advancing the face.

Heading 3 was driven about 7,800 ft. easterly from Shaft 1 wholly through the metamorphics—quartzites alternating with thinly bedded sandy shales predominating. Through this section numerous intrusions of granitic rocks (diorites, porphyries and related rocks) were encountered.

Each contact was accompanied by much faulting and weathering with the feldspathic minerals in the rocks in many instances highly

kaolinized, causing the ground to squeeze from the sides and arch and raising of the floor. The core of some of the intrusives, especially the diorites, was often very hard, sometimes requiring 60 per cent powder to break the rock. It also broke very blocky making mucking slow, often requiring secondary shooting.

In driving Heading 4, westerly from Shaft 2, various kinds of formations were penetrated. Starting at the shaft, excavation was in good hard basalt for a few hundred feet, then through volcanic agglomerates for a short distance when unconsolidated sediments, mostly sands and silts were encountered, some of these were very wet and heavy. Andesitic agglomerates followed the sediments, then good sound andesite for about 2,000 ft. While the latter stood well without support, large flows of water were encountered necessitating grouting.

Beyond the andesite driving was through several thousand feet of glacial gravels, followed by about 6,000 ft. of granite and then the metamorphics. Both the gravels and granite were crossed by numerous faults and made large quantities of water. The ground was heavy and required heavy support. In spite of utmost care taken, several runs occurred in advancing through the gravels; one such, occurring about 300 ft. back from the face, caused the suspension of driving operations for several months. During this period, all the tunnel in heavy ground from the run to the shaft for about a mile was lined with concrete for greater safety.

Heading 5, driven easterly from Shaft 2, like Heading 4, alternated in basalts and unconsolidated sands and gravels. In crossing an ancient stream bed about 1,500 ft. from the shaft, the roof caved in, partially filling the tunnel with debris and flooding the tunnel and shaft, the latter to within 80 ft. of the collar. Work was suspended for about a month.

Formations in Heading 6 also alternated between volcanics, mostly basalts, and unconsolidated ancient lacustrine deposits, the latter consisting of beds of volcanic ash, near the portal, then beds of silts, sands and gravels. Some of the silts, partially consolidated, would have a tendency to squeeze and heaving of the floor would occur. Very heavy timbering was required here.

The tunnel is supported for 40,322 ft. or for 67.4 per cent of its total length. Of the distance timbered, 23.2 per cent was spiled, and 18.5 per cent or 7,480 ft. was advanced behind breast boards.

The maximum water flow for all headings was a little over 20,000 gal. per min. or 44 sec.-ft.

At the present time, the tunnel is making about 14,000 gal. of water per min. and Heading 1 is making between 450 and 500 cu.ft. of carbon dioxide gas.

Where heavy ground was encountered, the excavation was enlarged, materially increasing the thickness of the concrete lining. Large amounts of water still coming in the tunnel at the time the lining was being poured gave considerable trouble. Drains were placed in the floor, and, where possible, seepage from the sides and arch was lead to them by sheathing with light sheet iron, or handled by pipe drains through the arch forms. The sides and arch were poured after the invert. Both were placed with a pneumatic gun charged directly from the mixer in the tunnel.

The details of equipment and methods of operation have not been covered in this paper. Those interested may find a well written article descriptive of the tunnel in all its phases in the December, 1938, issue of the *Western Construction News*, written by Wayne W. Wyckoff, member of the A. W. W. A., and engineer of the project.

Safety Measures

In spite of large quantities of water and gas, and in the face of heavy, treacherous ground, the safety record on the project has been very good. During the last two and a half years, during which time over 25,000 ft. of tunnel has been driven and 40,000 ft. lined, not a fatality has occurred.

When carbon dioxide gas was first detected, a mine rescue crew was trained, organized and equipped and has been functioning ever since when emergencies arise.

Special safety equipment and appliances have been installed at strategic points in Heading 1 to take care of any emergency arising due to exposure to gas.

At Shaft 1 heavy reinforced concrete bulkheads with steel doors were constructed on each side of the shaft to protect it from flooding in case a heavy run of water or a power outage should occur. Adequate standby pumping and ventilating equipment was installed at all headings and shafts.

The project is being constructed by force account under the direction of H. A. Van Norman, Chief Engineer and General Manager, Bureau of Water Works and Supply, Department of Water and Power, with W. W. Hurlbut as Assistant Chief.

Water Districts Pro and Con

Alexander Russell

THE water district problem has been with us at Rochester, New York, since 1908, when the Penfield Water Commission was created. The development was slow, until the great suburban boom of the 1920's. This period ended in 1927, by which time we supplied nineteen districts in the towns of Greece, Gates, Brighton, Pittsford, Penfield and Irondequoit and indirectly (through other water districts) served four additional districts in three of the above mentioned towns and also in Ogden and Parma. Since 1927 there have been a great many extensions of existing districts and the population of these districts has multiplied several times. Water district revenues have been an important factor in the growth of this company. To illustrate this the following figures may be interesting. In 1910 our water district revenues were .6 per cent of our total; in 1920 our gross revenues had increased 108 per cent, while our water district revenues had increased 1085 per cent. By 1930 water district revenues had increased 886 per cent over 1920 and represented 13.36 per cent of gross, and by 1938 15.81 per cent of our gross revenues represented water district sales, in spite of rate reductions made to this class of consumer aggregating more than $18,000 per year. At the present time more than 20 per cent of total sales of water are sales to water districts.

As a general thing water districts are created to serve undeveloped suburban areas where the revenue in sight would not justify a private corporation investing the substantial amount necessary to pipe up the streets involved. The water district, being an instrumentality of the town, issues bonds to finance its construction, installs mains and erects hydrants in sufficient number for adequate fire protection and thus at one time completes its distribution system without

A paper presented on November 3, 1939, at the New York Section Meeting at Utica, N. Y., by Alexander Russell, Vice President and General Manager, Rochester & Lake Ontario Water Service Corp., Rochester, N. Y.

involving the water company in any substantial expenditure. One or more 6-inch connections are made to the water company's mains and 6-inch meters installed and entirely adequate service thus furnished at a minimum of expense to the water company. It often happens that as the water district grows it reaches other points where additional connections are made to the water company's mains with resulting reinforcement of pressure and more satisfactory service. One of the water districts we supply occupies a large area in the town of Gates. We originally supplied them through an 8-inch main on Lyell Avenue at the north end of the district. As the district grew a second connection was made on Chili Avenue (the south end of the district) and a few years later a third meter was set in the middle of the district's east boundary on the Buffalo Road. Not only do these additional connections supply more water and higher pressures, but their existence enables the periodic testing of the meters involved without any appreciable reduction in service or the expense incident to installing bypasses around the meters, which construction is necessary when the district has only one connection with the water company's system. The water districts we serve are all in residential areas and the revenue is, in reality, from wholesale, domestic and commercial sales. One obvious advantage of this situation is that this class of revenue is depression proof. A disadvantage is that maximum demands always occur during a drouth when our own demand is at peak level. A further advantage of water district business is that there never is any question about collecting the bills rendered.

Summary of Advantages

To sum up the advantages to the water company of water districts, it should be noted:

1. Such districts make possible the extensive development of desirable areas.
2. Such development is carried out without the water company having to make extensive and expensive investments.
3. The revenue received is not affected by depressions or business reverses but increases steadily year by year.
4. Bills rendered are paid in full promptly, certainly and with no collection costs such as invariably attend all domestic sales.

There are always two sides to any question, not necessarily only the right side and the wrong side, and there are a number of valid

arguments against the establishment of water districts in suburban areas. Whether or not these drawbacks outweigh the advantages already referred to is a moot question. Let us look at the disadvantages of water districts.

Water districts increase taxes for a considerable period of years, not only imposing a serious burden on the owner of property in such areas but concealing in taxes the real cost of water service and fire protection. This is particularly true where the district does not build up rapidly and as a result the annual expense of bond interest and retirement falls with crushing force on the comparatively few homes built in the water district and on the vacant lots therein. I know of one instance where the annual tax for water district purposes only was over $25 per $1,000. In another district practically all the vacant lots, which comprised more than 90 per cent of the land therein, are being or have been sold for taxes, which the original developer of the tract found it impossible to pay. The most serious objection of all is, however, that the home owner is deceived into believing that his water service costs only the amount of his water bills; whereas, a large portion of such costs is buried in his taxes.

Again, water district charges not being subject to the control of any regulatory commission are high; they are not uniform; and they have no direct relation to the cost of furnishing service. In short, they are unscientific, discriminatory and unsatisfactory. This situation is illustrated by the fact that no two of the twenty-three water districts we serve directly or indirectly have the same rules, regulations, minimum rates, etc.

Third, each water district is a separate entity with clearly defined boundaries. Thus there is no interchange of water between districts and each becomes, in effect, a large dead-end area with resulting poorer residual (i.e. fire) pressures and quality of water than if the area was an integral part of the water company's system.

Fourth, the proper operation and maintenance of a distribution system can be carried on only by an adequate force of trained employees, with proper tools and equipment. None of the water districts in our territory can afford the large outlay necessary to meet these requirements and the result has been either careless and superficial handling of the district's affairs or the creation of expensive organizations which add materially to the cost of supplying water. An instance of the first kind of handling was disclosed when the

supervisor of a certain town sent for me and said, in effect, "We haven't the slightest idea of where the ———— Water District stands. No bills have been rendered to the consumers in nearly two years. There are no records of meter readings, etc., which we can rely on. For Heaven's sake take the District over and run it for us." We found the records to be audit proof because there were none! We sent a crew out to call at every house, secure meter number and reading, name and address of owner, lot number and house number and proceeded to make up a complete set of records from scratch. One interesting development was the discovery that Mr. Blank was getting water service without a meter and we found, upon running the matter down, that six years before he had given the district a rubber check in payment of the bill for the cost of his tap and that, therefore, the district had not bothered to set a meter so he had enjoyed unlimited free water for six years.

In another town we were asked to take over the operation of the ———— Water District and found the only record consisted of a ten-cent cardboard covered book in which the amounts of the bills, which were supposed to have been rendered, were entered, without any meter readings or consumption being shown, and the amounts paid were entered on the same line without any dates of payment. In both these districts we, of course, refused to accept any responsibility for the past, set up proper records and put the district's house in order, with a remarkable increase in revenues and reduction in the amount which had to be added to the tax levy each year.

Fifth, the very fact that the entire cost of establishing a water district and later making additions thereto can be financed by the issue of tax exempt bonds has been responsible for putting in mains and erecting hydrants in many streets which are still innocent of any houses. We all know how much more likely we are to spend money if some one else is footing the bill, or at least if we think they are. I believe it is unmoral and indefensible to spend money without the reasonable certainty of its earning a fair return. No private water company can do it and it is wrong to fall back on the power to tax—which as John Marshall once said "is the power to destroy"—to finance speculative ventures in subdivision development.

Finally, the creation of water districts, occupying as they do many of the better sections in the franchise territory of the water company, acts as a present revenue producer but future detriment to the water company's growth. As an example of this, let me outline what

happened in one township we serve. An important village, located about ten miles from one of our larger transmission mains, had a totally inadequate and unsatisfactory supply of water. We offered to extend a 12-inch main along the main highway (an important state road) leading to the village but the negotiations fell through. The village spent several hundred thousand dollars developing its own supply and nothing further happened until years afterward when a tentative suggestion was made that they might be interested in securing water from us. In the meantime, a water district was created to serve the people living on the main highway in question and about four miles of 8-inch and one mile of 6-inch pipe laid. Later the 6-inch line was extended about one and one-half miles further to supply another small water district. The result is that if we should now lay a 12-inch main on this road we would have nearly seven miles of it through absolutely non-productive territory and we were counting on the revenue to be received from the two hundred or more houses on this seven miles to help carry the line.

The above is not an unusual case. The water districts we supply stretch for miles in areas which otherwise would be our normal and proper field for service. Of course, some of these areas wouldn't be served now if we had had to find money with which to install mains therein.

In conclusion, in cases where extensions can be installed by the water company with the expenditure financed by the subdivision developer (who should be expected to carry the load) water districts should never be created, but where water service must be provided and it is not feasible on any other basis, then the water district is the logical instrumentality to use.

New Law on Water and Sewerage Districts

By William B. Cheek

ON MARCH 25, 1939, Colorado Senate Bill No. 367, by Senator Johnson and Representative Pearson, was approved. This bill provided for the organization of water and sanitation districts in Colorado and defined the purpose and powers thereof.

The purpose of the bill is to enable the taxpaying electors of a locality to create a water district, a sanitation district, or a water and sanitation district to serve their needs, and to extend water lines to the source of supply and sewer lines to an appropriate outlet and provide necessary treatment of sewage. The district can be created irrespective of county or municipality lines or boundaries. The bill empowers the district court sitting in any county in this state, or any judge thereof in vacation, to establish districts which may be either partially or wholly within the judicial district in which the court is located.

Organization of a district shall be initiated by means of a petition setting forth the name of the proposed district, the description of the improvements, the estimated cost of the proposed improvements, a general description of the district, and a prayer for the organization of the district. *The petition shall be signed by not less than 10 per cent of the taxpaying electors of the district and shall not include in the district any single parcel of property containing twenty acres or more without the consent of the owner or owners thereof.* It is further required that a bond sufficient to pay all expenses connected with the proceedings be filed to insure payment of those expenses in the event that organization of the district is not effected.

Upon filing of the petition, the court or a judge thereof in vacation shall immediately fix a place and time of hearing on the petition,

A paper presented on September 19, 1939, at the Rocky Mountain Section Meeting at Denver, by William B. Cheek, Civil Engineer, Denver.

not sooner than 20 days nor later than 40 days after the petition is filed. The clerk of the court shall cause notice by publication of the petition pending and the time and place of hearing, and shall send a copy of the notice by registered mail to the Board of County Commissioners of the county or counties having territory within the district and to the governing body of each municipality affected.

On the day fixed for the hearing or at an adjournment thereafter, the court shall determine from tax rolls, the last official registry list, and from other evidence, the total number of taxpaying electors residing within the proposed district. If the petition has not been properly signed and presented, the court shall dismiss the proceedings and assess costs against the signers as deemed equitable. Such action does not prevent filing of a subsequent petition.

Any time after the filing and before the hearing of a petition, the owner or owners of any real property in the district may file a petition to exclude such property from the district. The court will hear the exclusion petition and any objections thereto at the time of the hearing on the petition for organization, and determine whether the property shall be included or excluded from the district.

If a petition has been signed and presented in conformity with this Act, the court shall order that an election on the question of organization of the district be submitted to the taxpaying electors of the district. The election shall be conducted in the same manner as general elections in this state. Those voting for the organization shall vote for three taxpaying electors of the district for the first board of directors of the district.

If the majority of voters favor organization, the court shall declare the district organized, give it a corporate name, and designate the board of directors elected, and the district shall be a governmental subdivision of the state of Colorado with all the corporate powers of a public or quasi-municipal corporation. The salary of each director shall not exceed $60 per annum. The Act further describes the powers and duties of the board of directors, organization of the board, its meetings, election, and general powers. It also describes the financial administration as to taxes, levy and collection of taxes, levies to cover defaults and deficiencies, sale for delinquencies, and creating a reserve or sinking fund.

The Act provides by court hearings for changing the boundaries of the district after it has been organized as to either inclusion or

exclusion of real property and liability of such included or excluded property.

This Act will enable districts adjacent to municipalities, to incorporate for the purpose of floating bond issues in order to construct water and sewerage works systems. Oftentimes, a district is unwilling to join a municipality on account of taxes and by being able to form a sanitary or water district, the village eliminates the expense of a city government.

Those responsible for submitting this bill to the legislature were a group of property owners living in and near the Lakewood District, which is located west of Denver. The majority of this district have available the Denver city water, but do not have sewers, and because of the adobe soil, it is impossible to dispose of sewage properly by leaching in the ground with cesspools. The district has built up in the last few years very rapidly and is developing into a first class residential district. In the spring and early summer, however, the ground becomes saturated with water, and cesspools overflow almost continuously, thus creating a serious health menace, especially during the fly season, in addition to causing odors and unsightliness which depreciate property. Many of these home owners have spent much more for their individual sewerage systems than they would for their share of a modern, municipal sewerage system, including a sewage treatment plant.

Discussion Regarding Specifications for Deep Wells and Deep Well Turbine Pumps

By James C. Harding and J. Arthur Carr

Specifications for Construction of Deep Wells

By J. C. Harding

THE proper construction of deep wells is of vital interest to us because there are more public water supplies derived from this source than from any other.

I have recently made a survey to determine the extent of the use of well water in this country and, based on answers obtained from the health departments of forty-five of the forty-eight states, I estimate that of our total population of approximately 130 million persons, about 87 million are served by public water supplies and about 22 million of these users are supplied with ground water.

There are almost 13,000 public water supplies in this country, of which about 8,400 or 65 per cent are derived from ground water. It is true that all of the supplies classified as ground water are not obtained from deep wells, as particularly in the rural areas, quite a number of communities are furnished with water from shallow wells, springs and infiltration galleries. Deep wells do, however, represent the majority.

Despite the fact that such a large percentage of our population is served with well water, there seem to be very few technical data

A paper presented on June 13, 1939, at the Atlantic City Convention by James C. Harding, Consulting Engineer, New York and Mt. Kisco, N. Y.; and by J. Arthur Carr, Superintendent, Water Department, Ridgewood, N. J.

Editors Note—The Water Works Practice Committee has considered the material presented by Messrs. Harding and Carr and has authorized the organization of a committee on "Specifications for Deep Wells and Turbine Well Pumps."

The work of the committee under the co-chairmanship of Messrs. Harding and Carr is now in progress.

TABLE SHOWING POPULATION SERVED BY PUBLIC WATER SUPPLIES AND BY WELL SUPPLIES IN THE UNITED STATES

STATE	TOTAL 1930 CENSUS	SERVED BY PUBLIC SUPPLIES	SERVED BY WELL SUPPLIES
Alabama	2,646,248	1,067,506	238,264*
Arizona	435,573	224,111	182,110*
Arkansas	1,854,482	516,009	252,411
California	5,677,251	5,000,000	2,000,000
Colorado	1,035,791	603,902	68,264
Connecticut	1,606,903	1,483,500	32,400
Delaware	238,380	160,000	40,500
District of Columbia	486,869	650,000	—
Florida	1,468,211	969,700	874,793*
Georgia	2,908,506	1,214,500	460,000
Idaho	445,032	152,722	70,322*
Illinois	7,630,654	6,065,858	1,307,203
Indiana	3,238,503	2,014,000	970,000
Iowa	2,470,939	1,183,124	696,364†
Kansas	1,880,999	986,169	535,052*
Kentucky	2,614,589	967,258	122,289
Louisiana	2,101,593	969,850	274,700†
Maine	797,423	511,732	34,270
Maryland	1,631,526	1,205,656	105,000
Massachusetts	4,249,614	4,247,000	1,000,000*†
Michigan	4,842,325	3,644,506	859,843
Minnesota	2,563,953	1,514,000	456,000†
Mississippi	2,009,821	500,000	355,000
Missouri	3,629,367	2,133,089	307,192
Montana	537,606	249,199	40,979
Nebraska	1,377,963	775,093	531,014
Nevada	91,058	56,016	42,719*
New Hampshire	465,293	465,300‡	95,700†
New Jersey	4,041,334	3,920,000	1,770,000*†
New Mexico	423,317	169,986	126,161*
New York	12,588,066	12,035,000	1,513,000†
North Carolina	3,170,276	1,014,803	166,713*
North Dakota	680,845	183,643	115,147
Ohio	6,646,697	4,874,282	1,295,733†
Oklahoma	2,396,040	1,128,543	389,644†
Oregon	953,786	608,000	53,800*
Pennsylvania	9,631,350	8,000,000	296,000
Rhode Island	687,497	668,177	38,927
South Carolina	1,738,765	509,000	153,000

* Includes some population served by wells and other types of supplies.

† Includes some population served by other types of ground water supplies.

‡ Includes summer visitors.

TABLE—*Concluded*

STATE	TOTAL 1930 CENSUS	SERVED BY PUBLIC SUPPLIES	SERVED BY WELL SUPPLIES
South Dakota	692,849	250,000	200,000
Tennessee	2,616,556	1,042,602	421,487*
Texas	5,824,715	2,929,197	1,614,689
Utah	507,847	369,586	164,826*
Vermont	359,611	186,735	64,327*
Virginia	2,421,851	1,078,596	83,700
Washington	1,563,396	1,200,000	223,000
West Virginia	1,729,205	741,627	181,623
Wisconsin	2,939,006	1,795,004	764,024
Wyoming	225,565	130,428	70,577*
Total	122,775,046	82,365,009	21,658,767

The information contained in the above table was gathered by the writers in connection with an investigation recently made by them to determine the extent to which well water was used as a source of public water supply in the United States. With the exception of three small states, the figures given are based on information contained in letters and statistics received from the health departments of the various states listed.

The writers do not hold this table to be strictly accurate but they do feel that it represents the closest approximation that is available. Substantial difficulties were encountered in compiling the data because reports from some states are based on the population as given by the 1930 census while for other states they are as of different dates from 1931 to 1939. A further difficulty was that many municipalities were listed as having both well apd other types of supplies, the relative importance of which was not disclosed.

From the figures contained in the attached table and other data accumulated in the course of our investigation, it seems probable that if the present population of the country is assumed at 130,000,000—a figure frequently used—approximately 87,000,000 or 67 per cent are served by public supplies and about 22,000,000 people or 25 per cent of the total number of consumers are supplied with water derived either entirely or largely from wells.

Whereas only about 25 per cent of our public water supply consumers use well water, the number of well supplies far exceeds those of all other types. The total number of public water supplies according to our survey is 12,984 and of these 8,407 are derived from underground sources, the largest percentage of which are from deep wells.

generally available in convenient form that is helpful to those who draw specifications governing the proper construction of wells and there is now no single well specification in common use.

It is true that articles have been written that describe the experiences of various individuals with the construction of a particular

well. Most of these, while interesting, are not particularly useful as a guide in preparing specifications because they describe a well or well field that has been unusually productive or where the well construction has been unique.

There have also been a large number of papers published, descriptive of underground water bearing formations but these papers do not satisfy the needs of the water works engineer or operator who has to put a well in a particular place and is primarily concerned with how to construct the well, rather than with the name and general characteristics of the formation from which is to come the water he wants.

The water works man who turns to his textbooks for assistance in the preparation of his specifications will generally be disappointed if he expects to find anything very concrete. The bulk of the information contained in most textbooks relates mostly to the history of well drilling and to the method employed.

A few of the State Health Departments, notably that of the State of Wisconsin, have done excellent work in establishing rules and regulations governing the drilling of wells and they do, to a certain extent, serve as specifications. Most of them do not, however, answer the need completely because they are concerned primarily with the protection of the well water from pollution and not with the other important factor, that of water production. A further disadvantage is the fact that the average member of our Association interested in writing a specification for wells does not know that Wisconsin, for example, has particularly good rules and regulations unless he happens to live in that state.

The American Association of Water Well Drillers, realizing that the proper construction of wells was very important to its members, appointed a committee, which in 1935 brought out a standard set of specifications for the construction of both rock and sand or gravel wells. While these are of real value to anyone writing specifications for well work, it is felt by many of the well drillers themselves that they should be revised and amplified. However, apart from the merits of these specifications, there is a disadvantage in the fact that most members of the American Water Works Association do not even know that they have brought out such specifications. Also the task of writing specifications is primarily that of the consumer and not that of the vendor.

Improper Construction Hinders Development of Underground Supplies

It is rather difficult to prove that the lack of adequate specifications has affected adversely the development of underground supplies because naturally very few of us go around writing about or advertising our own mistakes or those of our friends. Nevertheless, those most in a position to know seem to agree that poor well construction is and has been a common occurrence. Almost any water works man can cite instances where to his personal knowledge an economic loss has resulted from an improperly drilled well.

Even well drillers who normally would be expected to be the last ones to complain because of the frequent construction of inferior wells realize the need for improvement. Apropos of this, Roy S. Barker, a veteran well driller, in a recent article says, "During the past thirty years, thousands of new drillers have entered the field, many of them without sufficient experience, or without capital to equip themselves, some careless, some downright dishonest, perhaps, in their advice to the inexperienced water user, who knows very little of well drilling. Few indeed, are the Engineers, fitted to give advice or to check on results."

The poor conditions now existing are also brought out in a letter written to me by a pump manufacturer in which he says, "While we do not deny that there have been many cases of operating difficulties with deep well turbines, we do feel that a vast majority of these troubles were caused by attempting to operate the pumps in crooked wells."

Another equipment manufacturer and an authority on well construction, in answer to a letter from me requesting among other things his ideas on the desirability of standard specifications for deep wells and deep well pumps, states "Just day before yesterday we were discussing this very matter—trying to determine just how consulting engineers in the country could be approached on this subject. By and large the profession does not know as much as it should in regard to the proper construction of water wells. All too frequently the specifications brought out on water well jobs make a mess of things."

Standard well specifications would not insure first-class construction of wells any more than our standard specifications for cast iron pipe insure the proper installation of cast iron pipe lines, however,

they would certainly help. With them the average water works engineer and operator would be able more intelligently to guard against the use of improper drilling methods, secure better construction and fairer competition, which should result in increased and more economical water production and better protection from pollution.

Standard specifications would be of great help to the water works man who has to prepare specifications for his new wells. As cited previously, the man who is writing a set of well specifications for the first time usually has limited sources of information and even the man who has drawn specifications governing previous well work is considerably handicapped because he must draw largely on his own experience.

Many wells are drilled without any specifications at all and a large percentage of the balance are the work of the local well driller and represent largely his ideas. It is certainly advantageous to consult with well drillers before drawing specifications, but there are a great many unreliable well drillers and even a first-class well driller is usually not any more competent to draw good specifications governing well construction than a first-class builder is to draw good specifications for a distributing reservoir. Also, depending too much on any one well driller tends to limit fair competition.

If this Association were to adopt standard specifications governing the construction of deep wells, it would be of assistance to the various state agencies having supervision over water supplies. The writer has letters on file from 27 of the state health departments representing 61 per cent of the total population of the country, which letters all state in substance that they consider that standard specifications would be of very material value to them.

Standard Specifications Can Be Prepared

There is no question but that it would be just as impossible to prepare well specifications that would fit 100 per cent of all well jobs as it would be possible to prepare pipe laying specifications that would fit 100 per cent of all pipe laying jobs. Each individual well installation requires careful study and separate and special treatment. It should, however, be possible to prepare specifications that would be broad enough to cover the vast majority of well installations, which with slight alterations or supplements would be of value in the preparation of specifications for the remaining well

installations even though it became necessary to deviate from the standard form.

The principal difficulty to be conquered is the broadness of the well drilling field. For example, deep wells in common use vary from 2 in. to 36 in. in diameter, from depths of a few feet to as much as 1,500 feet and have yields of from 5 g.p.m. to 5,000 g.p.m. Any standard specifications will require modification in many cases particularly when unusual circumstances are encountered, however, it is an axiom in the construction industry that no perfect specification was ever written for any construction job.

What Standard Specifications Should Cover

What any standard specification adopted by this Association should cover is a matter that will merit considerable study by experienced representatives of the water works profession including the well drillers and from all parts of the country. It is entirely possible that certain aspects such as the development of wells, type and size of screens, etc., should not be included, however, there are many factors such as the testing of wells, plumbness, alignment, casing, sealing, sterilizing and form of well records that certainly could be covered.

Conclusion

In view of the facts that such a large portion of our public water supplies are derived from wells, that so many defective wells have been and are being built, and that adequate specifications governing well construction are not readily available, it is my opinion that this Association, which more closely represents the water consuming public than any other organization, should take all steps within its power to insure the safe and efficient construction and use of wells. If the preparation and adoption of standard specifications governing well construction will help to accomplish this end, the necessity for prompt action on our part is clearly indicated.

Standard Specifications for Deep Well Pumps

By J. A. Carr

Mr. Harding has presented many good reasons why this Association should take the necessary steps leading toward the preparation and adoption of Standard Specifications for Wells. It is advisable also to discuss the necessity for standard specifications for deep well pumps. The two subjects are closely related and should be considered at the same time. Without a properly constructed well, and the information made available through the proper testing of the well, it is not only difficult but practically useless to prepare specifications for a pump for that well which will insure the obtaining of a first-class pumping unit. Conversely, unless the proper pump is selected all the care and thought used in constructing the well may be wasted.

Present Practice Described

To my knowledge there is at present no standard specifications of this Association, or any other technical group to which one can turn for guidance.

It is granted that Bulletin "E" of the Standards of the Hydraulic Institute contains much helpful data in reference to deep well pumps, but it is not strictly a specification. Moreover, the Hydraulic Institute is an association of manufacturers and possibly does not comprehend all the view points of the purchaser.

Twice within the past year I have had specifications returned by two manufacturers advising that they could not bid as certain things were specified which were not permitted by the Hydraulic Institute. Still eight to ten other manufacturers accepted the specifications without question, and not only submitted bids but were eager for our business.

It is my opinion, based on the perusal of many specifications, that the average water works operator, or engineer, about to prepare specifications for a deep well pump either picks up a pump manufacturer's catalogue or calls on a salesman friend for assistance. As a result the specifications are almost entirely taken up with the

dimension and material phases of the design of the pump, and many items which are vital to the proper evaluation of first cost and economical operation are omitted.

I recently read a set of specifications prepared by a leading engineer. There were many paragraphs devoted to adequate "this" and suitable "that," but the only paragraph of any importance said that the pump should be capable of discharging 100 gallons of water per minute against a total head of 150 feet from bowls being located approximately 90 feet below the ground. There was nothing regarding efficiency, or other pertinent and necessary information. One paragraph did, however, go into great detail as to how the pump head and motor should be painted.

Specifications Are Needed

It is my firm opinion that standard specifications for wells and deep well pumps should be prepared and that they would be helpful and welcomed by well drillers, pump manufacturers, and most of all by water works operators and engineers.

I have seen many instances, especially in small communities, where the pumps are entirely unsuited to the well conditions, and most uneconomical to operate. Had standard specifications been available for the men in charge of these plants to use, in all probability, many such faulty installations would not have occurred.

Too often all the necessary information regarding the characteristics of the well are not furnished to the manufacturer by the buyer. Also too often the information furnished to the purchaser by the manufacturer is not as complete as it should be and is presented in a form not comparable with that furnished by his competitor. This fact was well brought out by Mr. A. O. Fabrin, Chief Engineer of Layne-Bowler Company, who, speaking at a recent meeting of the Kentucky-Tennessee Section (A. W. W. A.)* said: "Recent tests and investigations indicate that the fault is not all that of the pump whenever field performance varies greatly from factory guarantees."

The manufacturer is frequently required to bid when the information furnished him is too meager or is erroneous and, after the installation of his product, is held responsible for conditions over which he had no control. With standard specifications for deep well pumps that condition could be avoided and much unnecessary controversy could be eliminated.

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* Jour. A. W. W. A., 31: 1675 (1939).

That standard specifications are needed I know from my own experience. For many years I have made a study of deep well pump specifications. I have tried to prepare specifications which would place all the necessary information before me on a basis which would permit me to select the unit best fitted for my needs, and still be fair to the manufacturer. I know I have succeeded in securing better efficiency and more economical operation. However, I still find it advisable to make changes in my specifications each time a deep well pump is purchased, in order to eliminate or clarify items which appear to be inequitable or liable to lead to disagreement.

A General Specification Can Be Prepared

It is fully realized that one specification for deep well pumps, one which would fit all conditions would be impossible to prepare. It is possible, however, to prepare standard specifications of a general character which can be readily adapted to specific cases.

In his talk before the Kentucky-Tennessee Section, Mr. Fabrin also said: "Specifications that pertain only to the pump which is to be installed in a well ofttimes are taken up solely with details of pump dimensions. . . . These proportions should be left to the manufacturer." This statement is quite true and in advocating standard specifications for deep well pumps there is no intention to enter into the matter of design.

There are, however, many things to be considered in the purchase of a deep well pump other than details of pump design that could be covered by standard specifications, and which are of much value and importance to the purchaser in judging bids which are received.

What Standard Specifications Should Cover

It is not intended to go into detail at this time as to what can and should be comprehended in standard specifications, but merely to bring out the need for a general and broad specification, equitable to all concerned, which can be used as a pattern and guide by water works men.

In general, standard specifications for deep well pumps should cover three major phases and should provide a method of clearly and uniformly stating all information that is to be given or received.

First—Information for the bidder. This should be supplied by the purchaser and give full information in reference to the well in which the pump is to be installed; such as type of well, diameter

and depth, static water level, expected draw down, and any peculiarities of the well. Under this heading should also be set forth information as to requirements of the pump itself; such as capacity, depth of setting, head against which the unit is to deliver, power conditions, installation, wiring and any accessory equipment to be furnished. The purchaser should also state his requirements or preference for materials to be used in the manufacture of the pump.

Second—Information to be supplied by the bidder. Under this heading the bidder should be required to furnish, on a standard form supplied by the purchaser, complete detailed data relative to the construction of the pump he is offering; such as size and weight of discharge column, size and material of shaft, bearing spacing, size and type of bearing, size and material of bowls, number of stages and numerous other details.

Third—Efficiency and Tests. To the purchaser of a pump the cost of pumping over a period of years is a prime consideration, other things being equal. Therefore, the efficiency of the unit offered is a major factor in evaluating bids.

Testing for efficiency and the method of test are of real importance. All too frequently pumps are purchased and installed with no adequate test after installation, a fact which may lead to over-optimistic guarantees.

If the efficiency of the unit is to be guaranteed, there enters the question of when it shall be tested, how it shall be tested, what instruments shall be used in the test and the method of calculation.

All these things and many more can be covered in a Standard Specification for Deep Well Pumps and covered in a manner fair to all concerned.

Benefits to be Derived

The adoption of standard specifications for deep well pumps would be equally beneficial to water works operators and manufacturers.

The benefit to the manufacturer would consist in his having an open specification through which he can place his product before the purchaser on a uniform standard basis easily comparable with that of his competitor. Rash claims by irresponsible bidders would be prevented.

That such rash claims are made by manufacturers seems to be borne out by the following quotation from a letter received by me from the president of a concern engaged in the manufacture of pumps.

"It was entirely right that you should have viewed our pumps from a critical standpoint until our promises were actually proved in our performances. We have gone to great lengths to make our performance guarantees absolutely accurate. We regret to say that some in the pump industry have not done this and there are always some wild guarantees floating around which, if accepted at their face value, would work great injury to those who make guarantees based only upon fact. If all customers who demand guarantees on their pumps would insist upon these guarantees being met, it would place the pump industry upon a higher plane."

Certainly the water works man would be benefitted by having the information on all equipment offered presented in a uniform manner enabling him to evaluate his bids readily and permitting him to select the pump best suited to his conditions and most economical to operate. He would be better able to prove or disprove the claims of the manufacturer in a manner acceptable to both parties, thus avoiding many unnecessary disagreements, arguments and possibly litigation.

If standard specifications for wells and deep well pumps are to be prepared they should be drawn up by a committee from this Association collaborating with, among others, representatives from the American Association of Water Well Drillers and from the Hydraulic Institute, so that the final draft would be equitable to all concerned.

It is a matter in which the closest representative of the ultimate consumer, that is the water works operator should take the lead. The American Water Works Association can perform a real service to all engaged in the production of water from wells by the preparation and adoption of standard specifications for well and deep well pumps.

Main Cleaning

By William S. Staub

MAIN cleaning projects have been undertaken at three West Virginia Water Service Company properties in this state. This is the first adventure into this kind of main restoration and will serve as an experiment. Where mains have been cleaned, a definite improvement has been noted by increased flows and in cases of the transmission mains by a reduction in the pumping cost per million gallons.

The purpose of this paper is to present general ideas on main cleaning, tell why the mains were cleaned, what results were obtained, what it cost per foot to clean the mains, and to give a general description of the procedure.

In years gone by, when there was no water treatment or when the treatment of water was in its infancy, the water works operator was capable of producing only a palatable or semi-palatable water. The chemical characteristics were given little thought. If the water was fairly clear it was satisfactory. Therefore, the distribution system suffered, due to lack of knowledge of water treatment. The deposition of the solids and the tuberculation reduced the area of the pipe and reduced the amount of water that would flow through the pipe with the pressure available. Consequently, today, when there are complaints of poor pressure, or when there is an increase in fire flow demands, the operator finds himself faced with the problem of a main replacement. Although the exterior of the pipe is practically as good as the day it was installed, the interior presents a scene comparable to the stalagmites and stalactites of the eastern caverns. If this coating could be removed from the pipe and the carrying capacity restored there could be a great savings over replacement.

A paper presented on October 13, 1939, at the West Virginia Section Meeting at Morgantown, W. Va., by William S. Staub, Sanitary Engineer, West Virginia Water Service Company, Charleston, W. Va.

When a main is suspected of being tuberculated, or when it is found that the operating head on a pump is much greater than the static head, it is time to find the trouble. The distribution main is isolated in such a manner that all water going to a certain area must pass through the section of pipe to be tested. Then, in the early morning hours, the calibrated pressure gages are placed along the

TABLE 1

Charleston Main Cleaning

ITEM	PIPE 6-IN. KANAWHA ST.	PIPE 8-IN. VA. ST.	PIPE 10-IN.* SO. CHAS.	PIPE 10-8-IN.** NITRO LINE
Contract of Cleaning	$134.80	$271.70	$397.60	$1,488.95
Material Cost	6.19	48.86	138.29	250.40
Labor Cost	65.86	186.80	188.14	888.09
Total Cost	206.85	507.36	724.03	2,627.44
Number of Feet Cleaned	1,348	2,717	3,976	29,779
Cost per Foot	$0.1535	$0.187	$0.1823	$0.0834

* This cost does not include 8-inch temporary steel line which was laid. The rental of pipe, placing and removing cost $1,757.75 for 4,200 ft. If this were added to the above, the cost per foot for cleaning the 10-inch main would be $0.625.

** The 10- and 8-inch main is a transmission main and was cleaned as one unit; therefore there was no breakdown in cost.

TABLE 2

Charleston Main Cleaning

LOCATION	SIZE	FEET OF PIPE CLEANED	"C" BEFORE CLEANING	"C" AFTER CLEANING	EST. SAVINGS IN COST PER MILLION GAL.
South Charleston	10-inch	3,976	66	130	$2.37
Nitro—Dunbar	10-inch	10,679	49	130	
Nitro—Dunbar	8-inch	19,100	49	126	
Charleston—Virginia St.	8-inch	2,717		138	
Charleston—Kanawha St.	6-inch	1,348		115	

line and a hydrant is opened at the end of the line. The amount of water is measured with a pitot tube, the pressure drop is recorded and the loss per 1,000 ft. or the value of "C" is determined. We admit this method of determining "C" is not as accurate as secured by the pitometer, but we feel that a reasonably accurate value is obtained if precautions are used. In the case of transmission mains,

by determining the efficiency of the pumping equipment, static head, operating head, power consumed and water pumped, it is possible to get a very good check over any period.

A dirty main is definitely a liability. From the operating standpoint a dirty transmission main consumes many kilowatt hours of power due to the added friction. Kilowatt hours in turn cost money, which in turn increases operating expenses. A dirty main is a great handicap to the fire departments, due to insufficient quantities of water. Poor pressure complaints from consumers are not sooth-

FIG. 1. 8-inch Uncleaned Pipe

ing melodies to a superintendent's ears. Consequently, everyone suffers from the effect of tuberculated pipe. Thus, for the first time, we have ventured into a different method of improving service to our consumers, and at the same time bettering our operating conditions. This work has been an experiment to show us to what extent we can rely on this procedure for our future needs.

The largest pipe which we cleaned was 10-inch cast-iron. The 10-inch and 8-inch pipes were cleaned with the pressure machine which uses water pressure to push the machine through the pipe. This machine is about 6 ft. long, composed of five cutting heads and

TABLE 3

Total Cleaning Costs, Hinton Main

ITEM	10-IN. C.I.	8-IN. C.I.	6-IN. C.I.	4-IN. C.I.
Contract of Cleaning	$177.40	$546.90	$255.60	$675.80
Labor Costs	42.92	238.33	185.60	376.86
Material Costs	44.25	245.54	189.30	301.96
Total Cleaning Cost	264.57	1,030.77	630.50	1,354.62
Cost per Foot	0.15	0.189	0.246	0.20

TABLE 4

Hinton Main Cleaning

SIZE PIPE	FT. OF PIPE CLEANED	NO. OF "SHOTS"	MAX. LENGTH "SHOT"	MIN. LENGTH "SHOT"	AVE. LENGTH "SHOT"	"C" BEFORE CLEANING	"C" AFTER CLEANING
			feet	*feet*	*feet*		
10-inch	1,774	1	1,774	1,774	1,774		128
8-inch	5,468	9	1,072	215	608	48	126
6-inch	2,556	7	675	189	365	80	117
4-inch	6,758	14	965	36	482	44	122

TABLE 5

Sutton Main Cleaning

ITEM	PIPE 6-IN. MAIN ST.	PIPE 4-IN. TOWN HILL	PIPE 4-IN. SOUTH SIDE
Contract of Cleaning	$177.00	$325.40	$487.50
Material Cost	18.55	72.60	105.60
Labor Cost	90.50	166.40	249.20
Total Cost	285.85	564.40	842.30
Number of Feet Cleaned	1,770	3,254	4,875
Cost per Foot	$0.161	$0.173	$0.172

TABLE 6

Sutton Main Cleaning

LOCATION	SIZE	FT. PIPE CLEANED	"C" BEFORE CLEANING	"C" AFTER CLEANING
Main Street	6-inch	1,770	42	136
Town Hill	4-inch	3,254	68	118
South Side	4-inch	4,875	71	117

two pistons. The heads are "vertebrae" upon which are attached, around the circumference, the cutting blades. The blades are of spring steel. There are two flexible pistons from which the machine gets its drive. There are adjustable openings in the pistons, with which the operator can control the volume of water passing the machine to carry the debris ahead and at the same time prevent the machine from moving too rapidly. If the ports are closed added driving power will be given to the machine. The pistons and cutting heads are on a spring and flexible cable shaft. This flexibility will

FIG. 2. "Carrier" Used to Pull Cable through Pipe

allow the machine to pass a standard 90° bend. Contrary to opinion, the machine does not have any spiral or whirling motion.

The main is isolated and a section of pipe is removed at the point of beginning. This section should be long enough to give ample working room for inserting the machine. The machine is then jacked into the pipe and the pipe permanently jointed either by solid or Dresser sleeves. At the point which the machine is to emerge the pipe is again cut. A short section is removed and bell and spigot fittings are used to rig a riser. Usually this is done with a 45° bend and a short piece of pipe. The joint need only be juted and chained. The purpose of the riser is to deposit the debris out of the trench. When the riser is rigged the pipe is all ready. The valve on the main is opened slowly to secure the proper amount of

water and the machine begins to move. The machine generally moves about 100 ft. per min. and it is possible to follow the travel of the machine, as the water passing the pistons and the scraping blades is audible at the surface. Generally the water emerging from the riser will be only colored until the machine gets within about 300 ft., then the water is thick and black. The solid material issues from the pipe like sausage. In the case of the 3,900 ft. of line in South Charleston there was about one and one-half tons of solid

FIG. 3

FIG. 4

FIG. 3. 8-inch Cleaning Tool
FIG. 4. Solid Material Removed from 10-inch Main

material which came from the pipe. The machine will generally lodge in the riser pipe due to so much debris. When the machine has reached its destination the riser is dismantled, the main flushed and jointed and placed in service.

In the case of 6-inch mains, which are not so dirty, they may be cleaned by "pressure shots" such as in the larger sizes of pipe. In one case where the pressure was low a booster pump was used to supply increased pressure for a 6-inch "pressure shot," but generally

the dirty 6-inch mains are cleaned the same way as 4-inch pipe, as described below.

In cleaning 4-inch pipe the section to be cleaned is isolated and the main opened at both ends. At the beginning point a small cable is attached to a "carrier" and inserted into the line. The "carrier" is a short rod with leather pistons. The main is then jointed at this point with a portable coupling known as a "makeup." This "makeup" has a gland in the top through which the small cable may be fed. The water is then turned on behind and the water velocity takes the "carrier" and the small cable to the other end of the pipe. This small cable is then used to pull the big cable back to the point of beginning. The makeup is removed, the cleaning tool clamped to the large cable, the main permanently jointed, the wash water turned on, and the cleaning tool is pulled through the pipe by a gasoline driven winch. This process is much slower than the pressure method and great care must be exercised to prevent breaking the cable and loosing the machine. This winch automatically releases the clutch when the tension or pull becomes too great. In this case a very slow speed is used. This generally happened when the cleaning tool caught some lead or a bad obstruction. When the cleaning tool emerged the main was flushed and permanently jointed.

Obstacles Encountered

In pressure cleaning the drive of the machine is dependent upon the velocity of the water acting on the pistons and in the event the machine encounters a cross increasing on the branches it will generally stop. In other words, in cleaning an 8-inch pipe, if at an intersection a 10x8 in. cross is present, the machine will hang. This is due to the pistons expanding and allowing the water to pass instead of drive the machine.

If the wash water passing the machine is not sufficient to carry the debris ahead the machine will be fouled. One 8-inch main was encountered in which the machine would move a short distance, then stop. When the wash water would move the material ahead the machine would move. This happened on a long line where wash water was insufficient and blowoffs were too far apart. This main had to be recleaned and the writer is of the opinion that part of the debris was deposited behind the machine during the first cleaning.

Short radius bends with valves on one side caused several cuts to be made.

Insufficient pressure was a jinx on one job at Hinton where was being cleaned a 10-inch main from the reservoir down the hill. The pressure at the point of beginning was only 17 lb. Thus it was necessary to fill the line with water and bleed it rapidly at the bottom of the hill, thus creating a vacuum. When this was done it was impossible to keep up with the machine.

Poor joints were responsible for a number of worries—joints which were poorly juted and where the lead had run into the pipe. There were chunks of lead removed weighing 50 to 75 lb., old pump gaskets, coil springs, timbers, rocks and, in one case, a piece of $1\frac{1}{2}$-inch pipe about 6 ft. long. These had probably entered the system years ago when reservoirs were uncovered or when the pipe was laid.

At Hinton practically the entire system was cleaned. The method in which tie-ins were made, and the fittings and location of mains were definitely established. The superintendent there can now point out on the ground just what is at every intersection and he also made intersection cards accurately during the cleaning.

There were cases when a pipe which was supposed to be was not and vice versa. There were old valves found, box and all gone, stem broken, and found partly closed.

If anyone operating a water system which has been in service for a number of years attempts a job of main cleaning, he will probably be surprised at a great many things found (with apologies to those plants with excellently preserved records). And from the operating standpoint the knowledge gained during cleaning of mains is worth a few dimes and a few nights sleep to the superintendent.

Conclusions

It is felt that main cleaning has done a lot of good. Those who have seen the mains before and after know that flows have definitely been improved. Just how long this pipe will remain clean is another question. Some engineers maintain that the tuberculation will gradually build up. It is hoped that a close check on cleaned mains will be kept. Daily samples are now being taken at all points where mains were cleaned and records are being made of the pH, alkalinity and carbon dioxide. Periodic flow tests will also be made to ascertain to what degree the main is tuberculating. An attempt is being made to control the treatments accordingly.

It is known that fire flows have been greatly increased and that once weekly hydrants are again gushing briskly. On transmission mains there have been definite reductions in power consumed.

Observations on Water Softening

By Daniel E. Davis

THE seeds of most technological developments grow slowly. This has been particularly true in the field of water softening. Although the basic knowledge of the lime-soda and the zeolite (or base-exchange) processes has been a part of our technical stock for many a decade, the application of this knowledge in a large way to the softening of municipal supplies has awaited the turn of this century. The steps in the progress of this art can be traced with some assurance. The fundamental chemistry has not changed—lime is still employed in converting soluble bicarbonates to normal carbonate precipitates, soda ash in reducing permanent hardness, and salt in regenerating zeolites—but the mechanical procedures have undergone considerable advancement with the years.

One of the earliest large municipal plants of the lime-soda type was built at Columbus, Ohio, about 1907. Mixing was accomplished in vertical and horizontal baffled chambers followed by approximately seventeen hours of settling prior to filtration. Experience revealed that such a long settling period was unnecessary and unduly expensive. With the realization that the precipitates were heavy and settled out quickly, there was introduced the clarifier, usually of about one-half hour capacity for the continuous removal of the precipitates, followed by settling basins whose capacities were reduced progressively to eight hours, then to four and even less.

The practice of the thorough mixing of the chemicals with the hard water at a rate that would not break up the "floc" became established about 1925 with the introduction of mechanical paddles. Some of the earliest examples of these mechanical mixers are to be found at the South Pittsburgh Water Company, supplying suburbs

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south of Pittsburgh, and at Greenville, Ohio. This slow stirring gathered together the incipient precipitates and, by continued growth, built up a large floc which quickly settled out on reaching the clarifier. As a matter of observation and fact, two 20-minute mixing chambers connected in series and delivering to a 30-minute mechanical clarifier produced a crystal clear water in the latter. In these earlier clarifiers the means of taking off the clearest water had not developed to the point it has now achieved, otherwise an additional settling basin as such would not have been essential.

One of the troubles experienced at the earlier plants was the deposition of carbonates on the sand grains of the filters or softeners, and also in meters and domestic piping. At Defiance, Ohio, a method employed at Winnipeg, Canada, many years before, was reintroduced whereby carbon dioxide gas was pumped into the water thereby changing some of the normal carbonates to the soluble bicarbonate form.

A little later it was observed that if somewhat more lime were added than theoretically required, a greater removal of hardness was obtainable than was previously experienced, and that magnesium could be reduced to a low minimum. The excess lime renders the water caustic and thus facilitates the quick conversion of magnesium through the bicarbonate stage to a hydroxide precipitate. In some plants carbonation is practiced immediately beyond the clarifier. The causticity is changed to normal carbonates which settle out in part in the settling basin and the remainder is taken out on top of the filters. This procedure results in the reduction of carbonate hardness to a point close to the solubility limit, but there is a building up of lime in the sand. This can be avoided by carbonation just ahead of the filters, but at the expense of somewhat harder water.

In many plants this lime growth is progressive and not so much on the sand grains themselves, but as slabs or slivers of lime resembling plaster which even a high wash rate appears incapable of removing. This would appear to be an excellent job for the surface wash or the "filter sweep" which might make it possible to keep this building-up or growth wholly in check. Any normal carbonate which might slip by the filter and deposit in the distribution system can now be kept in solution by the "threshold treatment," whereby the application of sodium hexametaphosphate in doses as low as 1 p.p.m. is effective in preventing after-precipitations.

Sperry, at Grand Rapids, Michigan, nearly a quarter century

ago recognized the value of returned sludge in effecting more complete reactions. Laboratory results were successful, but the return of sludge on a plant scale broke up the floc to a milky consistency which proved almost impossible to precipitate. It remained for Spaulding to develop a design whereby the water with the freshly applied chemicals was compelled to travel through a "sludge blanket" thus achieving more thorough reactions without breaking the floc and with a reduction of total mixing, reacting and settling time to an hour or less. Some reduction in chemicals is reported, but excess lime is still required where magnesium is to be reduced.

In all this advance in the art the trend has been in the direction of reducing the time of the reactions with a consequent reduction in the cost of the plant structures, and a better control of softening operations so as to secure a softer water with low magnesium content not subject to after-precipitation. It appears that so far as the lime-soda process is concerned these objectives have been pretty largely attained.

First Zeolite Softening

The first large municipal zeolite softening plant (and until recently the largest) in this country was built in 1926 at the head of Neville Island to serve an extensive suburban area below Pittsburgh. In this process calcium and magnesium, the hardness-producing elements in the water, replace or exchange with the sodium constituent of the zeolite sand when passing through it, thus producing a soft water of zero hardness. When the sand becomes exhausted it is regenerated by passing a common salt (sodium chloride) solution through it when the opposite exchange takes place, the sodium entering the sand and the calcium and magnesium being flushed to waste to the sewer.

Green sand or glauconite is a natural product found along an ancient coast line in New Jersey and when sized and treated is ready for service. It has an exchange capacity of about 2,800 to 4,000 grains of hardness removed per cubic foot of mineral, while synthetic zeolites usually range from about 6,000 to 12,000 grains. Treated carbons have recently been developed which can be regenerated with acid. The plant design usually follows closely the typical rapid sand filtration plant with either downward or upward flow.

The pioneer units were of the closed design, thus following the earlier rapid sand filtration practice; but the tendency appears to

run to open or gravity layers in which the condition of the layer can be more readily checked and its better maintenance assured. Automatic operation is practiced in many plants, and the sequences of washing, brining, rinsing and softening are handled by electric relays.

Fortunately this base-exchange phenomenon is not limited to magnesium and calcium alone, and manganese can be removed from water in like manner, and water of zero hardness will also be free of manganese. Water containing this element in solution can also be "demanganized" by passing it through a zeolite which is regenerated by potassium permanganate.

Zeolite lasts for many years with but little loss through attrition. Usually a dropping off in exchange capacity is noted with the years, some of which is probably due to the deposition of an inhibiting film on the sand grains. Some, however, and possibly the major part, is probably due to the uneven washing of the sand layer because of "gravel hills" or "gummed up" areas. The water to be softened bypasses these areas and consequently the whole layer is not fully effective in softening. In some plants, holding-down screens over the gravel have kept the gravel wholly in its appointed place and the zeolite sand is completely and uniformly raised during the washing operation, and settles back in proper grading. The exchange capacity of such layers is surprisingly well maintained.

Reducing Salt Requirements

Since salt is the principal element of cost in zeolite softening it follows that means for reducing the amounts required should be at a premium. Running the zeolite at lower than rated exchange capacities appears to reduce the salt requirements. In one plant employing this method the salt dose has been approximately .34 lb. of salt per 1,000 grains of hardness removed, and often is as low as .3 lb.

Each method of softening has its own special place. The lime-soda method would usually be used wherever the raw water was contaminated or turbid, while the zeolite process is best adapted to clear, pure well waters. The zeolite process is usually cheaper in plant cost since mixing, clarifying and settling facilities are not required and the storage arrangements are cheaper than for the lime-soda type.

Several surveys have thrown light on savings to domestic consumers in terms of soap. Naturally these savings vary with the

hardness removed, and no general statement will cover all cases. In many communities it has been found that soap savings amount to 70 per cent of the soap cost under hard water conditions, and equal or exceed the water bill.

A comparison of the cost of chemicals for the two types of softening can readily be made for waters of varying hardness. In the preparation of the curves in Fig. 1, it has been assumed that the lime is quick lime or CaO of 90 per cent purity at a delivered cost of $12 per

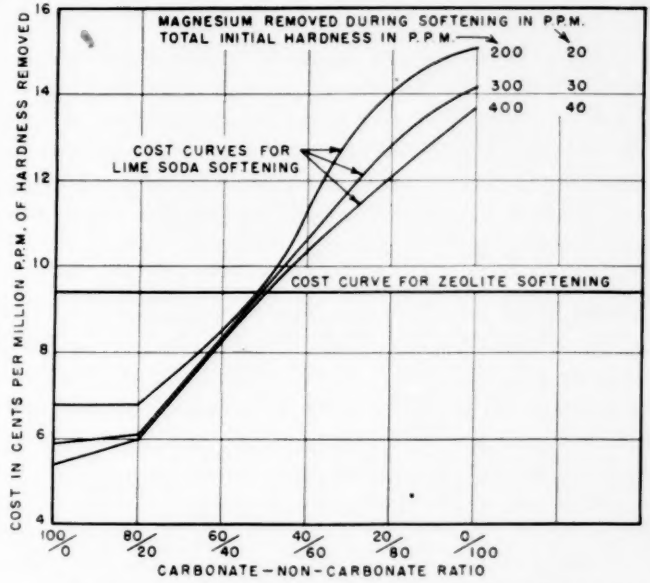

FIG. 1. Curves for Chemical Costs in Water Softening

CHEMICAL COSTS PER TON: Lime (90 per cent CaO), $12; Soda Ash (99 per cent Na_2CO_3), $26; Salt, $8.

ton, soda ash is of 99 per cent purity at $26 per ton and salt is available at $8 per ton.

It has been assumed that the water would be softened to 80 p.p.m., and that carbonate hardness would be reduced as far as practicable before soda ash would be employed in reducing the non-carbonate hardness. The lower limit for carbonate hardness removal has been fixed as 25 p.p.m. Separate calculations have been made for total hardnesses of 200, 300 and 400 p.p.m. of which the magnesium content removed in softening was assumed as 20, 30 and 40 p.p.m.

respectively, which correspond to many situations ordinarily encountered in practice.

The cost of zeolite softening is predicated on the use of .4 lb. of salt per 1,000 grains of hardness removed, which at $8 per ton represents a cost of $1.60 per million grains removed or a cost of $.094 per million p.p.m. removed.

The curves which depict the results of the calculations in graphic form reveal that so far as chemical costs are concerned the lime-soda process is less expensive when the non-carbonate hardness is less than 50 per cent of the total hardness, and that when the non-carbonate hardness predominates the zeolite is more attractive. This comparison leaves out of consideration the labor and fixed charge costs which would weigh the balance further in favor of the zeolite treatment. Each problem requires special investigation, but the art of softening is so well advanced that communities afflicted with hard water can proceed with assurance to obtain a softened product with the knowledge that the savings will exceed the cost.

Discussion by *H. M. Olson*.* There have indeed been many interesting developments in municipal water softening since its inception in this country in 1903 at Oberlin, Ohio. In the state of Pennsylvania there are at present a total of 42 municipal water softening plants of which 12 are of the zeolite type and 30 of the chemical precipitation type, an increase of 5 since April 1, 1938.

The largest chemical precipitation plant now in operation is supposed to be at Columbus, Ohio, although Minneapolis has been building for three years on a 120,000,000-gallon Spaulding precipitation plant which the author is told will be ready for use sometime in 1941. The largest municipal zeolite operation is, reputedly, at the Pittsburgh Suburban Water Company's Neville Island plant, McKees Rocks, Pa. It delivers a peak of 5,700,000 gallons distributed maximum, and removes approximately 40,000 kg. of hardness in one day, using 3,400 cu.ft. of greensand zeolite in the operation with brine obtained from wells for regenerating purposes. At Clarksburgh, W. VA., 5,000 cu.ft. of greensand zeolite is in use at one plant. At Maywood, Illinois two plants are in use, one with 3,000 cu.ft. and the other with 2,000 cu.ft. of zeolite; both of these cities delivering less water and removing less hardness than at Neville

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Island. The largest combination lime-zeolite plant is in use at LaGrange, Ill., where an average of 800,000 gallons of 53-grain water is treated by lime and upflow synthetic zeolite. And, the first unit of a four unit plant is now being built for the Metropolitan Water Board of Los Angeles, Calif., to treat Boulder Dam water of 19-grain hardness, capacity 100,000,000 gallons per day, using lime pre-treatment followed by sodium hexametaphosphate and upflow synthetic zeolite softeners.

In the chemical precipitation type of treatment, the sludge blanket systems as exemplified by the Permutit Company's Spaulding Precipitator and the International Filter Company's Accelerator are steadily replacing the old conventional design. Considerable work has been done in the matter of sludge removal in the interesting experiments by The Dorr Company with the Bird Centrifuge at Findlay, Ohio, and in other experiments on heavy sludge now being conducted. Carbonaceous zeolite employing the sodium cycle is used at Morocco, Ind. There is the possibility of this same type of zeolite being utilized with the hydrogen cycle acid regeneration for the reduction of total solids especially on hard waters or where considerable sodium salt is present in the raw water supply.

Corrosion Control

With the softening of water supplies the ever-present problem of corrosion is before us. The development and application of the Langelier formula with the new use of sodium hexametaphosphate to inhibit corrosion as well as to check after-reactions are finding favor; their merits are being recognized. Hoover is conducting experiments in hot water tanks with various amounts of sodium hexametaphosphate and should have some very interesting reports for us soon.

In automatic operation of softeners the soundness of the designs and construction has been proven with the large number of plants in operation and the really small amount of trouble encountered. The application of the electric eye to make periodic soap analyses on a softened supply and to actuate the regeneration cycle equipment is another interesting development which will find favor in time as its merits become known. It is very useful on waters having varying and quick changes in hardness when treated by the zeolite method of softening.

The many developments in other fields of treatment also have a great bearing on the softening problems, such as the use of sulfur dioxide for pH control, uses of various substances in coagulation, removal of gases and the continuous advances in control of bacteria. Another advancement is the application of sea water for regeneration of municipal zeolite units as at Sarasota and Hollywood, Fla., and also in many industrial plants in Japan. The sea water is coagulated, chlorinated and filtered. It is applied as a backwashing regenerating operation. One thing the writer has found lacking to the greatest extent is the keeping of accurate and adequate records which are valuable for study in locating troubles in operating and for contemplated changes and additions.

Development of Submerged Combustion for Recarbonation

By Leroy H. Scott

VERY little is to be said about the process of recarbonation, as this has been widely developed during the past twenty-five years, with the result that the subject has been well covered in many previous reports. However, few mechanical changes have taken place since the first installation. The equipment is still massive, costly to install and maintain, subject to severe corrosion, and difficult to control. The usual practice is to burn coke or gas, collect the carbon dioxide bearing products of combustion, filter, wash and dry these combustion products, and finally compress them for passage through submerged grids or gas diffusion apparatus into the water. It was with the idea of greatly simplifying this equipment that submerged combustion was developed for recarbonation.

In 1887, Oscar Brunler started work on submerged combustion, but it was not until 1913 that he was able to make a commercial installation. Oscar Brunler, Jr., took up the work after his father's death, and evolved the first working steam boiler using submerged combustion. Since that time, in this country, much work has been done by C. F. Hammond and Kemp, who have developed underwater burners, many of which are in successful operation. The Ozark Company, at Tulsa, Oklahoma, became interested and has developed its own burner which is much simpler than any developed in the past, due to elimination of any refractory in the combustion chamber. These burners have proven satisfactory and have been in use for several years in The Ozark Company's chemical plant, evaporating sodium sulfate.

The many efforts made since the first patent in 1887 show the struggle to discover all of the necessary conditions for submerged combus-

A paper presented on October 19, 1939, at the Southwest Section Meeting at Dallas, Texas, by Leroy H. Scott, Filtration Engineer, Water Department, Oklahoma City, Okla.

tion. Failure to employ all of these principles accounts for failure in operation.

Essential elements of the burner in which combustion takes place below the surface of a liquid are: (1) a mixing chamber from which a homogeneous mixture of air and fuel will result—the air and fuel must be delivered at a pressure to the burner above the submergence pressure; (2) a velocity tube through which the fuel-air mixture passes to the burner at a rate greater than the rate of flame propagation of the combustible mixture—this prevents the burner from back-firing into the mixing chamber; and (3) a burner so designed as to taper this velocity of fuel-air mixture and allow combustion to take place.

FIG. 1

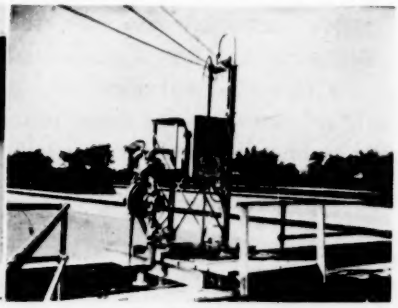

FIG. 2

FIG. 1. The Burner
FIG. 2. The Control Assembly

Due to the fact that combustion occurs within the body of the liquid, the products of combustion and the steam produced will leave together. This water vapor, carbon dioxide, oxygen and nitrogen are the gaseous products which all leave the burner together. In this mixture the molecules of each component gas are uniformly distributed throughout the entire volume. Each component gas will contribute a definite fraction of the total pressure. This part of the total pressure contributed by any one of the component gases is called the partial pressure. The total pressure is the sum of all the partial pressures. Due to this phenomenon, the gases of combustion are relieved from the burner in a very finely divided state, and absorption is almost immediate. This is the characteristic that seemed to make submerged combustion ideal for use in any recarbonation application.

With this idea in mind, the problem of recarbonation with the use of under-water burners was taken to The Ozark Chemical Company and enough interest aroused to encourage them to make some experiments at Oklahoma City to adapt their burner for recarbonation.

The first attempt was purely experimental, with rather crude apparatus, all of which was manually controlled, and though we were successful in burning gas under water, and actually recarbonating to some degree, the experiment was by no means an overwhelming success. Overall efficiencies were low—the apparatus was hard to control, and due to mechanical difficulties, not dependable. However, the success attained warranted further development, and with this in mind, a set of specifications was made, covering all of the desirable features that any recarbonation plant should have. Also a list of all the undesirable features in the then existing plants was made, and we next set ourselves to design one plant which, insofar as possible, would incorporate all of the best features, and at the same time eliminate all of the undesirable ones.

Submerged combustion of a fuel, free from sulfur compounds, appeared to offer possibilities of greatly simplifying recarbonation practice by making possible the use of automatic controls, and by removing the cause, and hence the effect of corrosion. There is no further need for the scrubbing and compression of the acidic carbon dioxide bearing gases. Also increased overall efficiency (the ratio of carbon dioxide utilized to that available from complete combustion of the fuel), with a decrease in fuel costs, seemed a probable effect to be gained by submerged combustion. The elimination of much of the equipment, such as scrubbers, driers, and diffusing grids, tends to reduce the size of the installation, with its necessary expense both as to installation costs and upkeep.

Due to the excellent absorption of the gases of combustion from the under-water burner, no diffusion grids are necessary. This makes the possibility of installing the complete recarbonation equipment in plants that are now in operation without the necessity of shutting down and draining any basins that are in service.

The design of the equipment finally evolved, and now working satisfactorily at Oklahoma City, requires that the sources of air and gas have pressures above that of the submergence pressure of the burner. Where natural or artificial gas is used, it can be had in most plants at a line pressure sufficient so that no compression is

necessary. However, in special cases this gas is put through a small booster in connection with the regular air compressor so that both the gas and the air ratio are maintained. The flow of air and gas at constant pressures leads to a fuel-air proportionery-mixer. This piece of equipment serves to compensate automatically the flow of one gas with any variation in the flow of the other—to maintain proper pre-determined proportions for complete combustion. The proportion or ratio of air to gas is regulated at the time of installation and need not be altered so long as the fuel remains reasonably constant in composition. The volume of fuel, and hence mixed fuel-air in the form of a correctly proportioned combustible mixture, may be varied by one valve control over wide limits to meet variations in water flow through the softening plant. The equipment is fully automatic, both as to change in rate and chemical composition

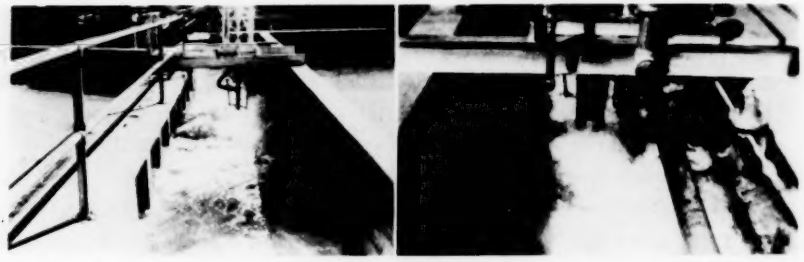

FIG. 3. Views of Channel in which the Burner is located, showing agitation resulting from recarbonation

of the water, being operated from a pH recorder-controller located downstream from the recarbonator.

From the proportioner the combustible mixture flows through a short horizontal line to the burner shaft and on to the submerged point of ignition. In case of flame failure, which may result from interruption in flow of gas or air, the control valve at the proportioner is automatically shut off by action of a solenoid valve, normally held open during operation of the burner by a thermo-couple in the combustion zone of the burner. At the same time this sounds an alarm in the building, notifying the operator that the burner has failed due to either the failure of the air or gas supply. Ignition of the burner is electric, and it is only necessary to reset the solenoid valve and press a switch to resume operation.

The burner in this installation consists of a cylindrical section of

suitable diameter, open across its entire section at the bottom or discharge end, and equipped with a frusto-conical expansion chamber at the upper end. This expansion chamber is normally threaded at a point near the cone apex to allow union with the mixed gas feed line. The capacity and attainable variation of a burner of fixed size is determined by various details of construction. The burner now in use at Oklahoma City is five inches in diameter and has a maximum capacity of two thousand cubic feet per hour of natural gas with a B.t.u. of one thousand.

Combustion in the submerged burner takes place at temperatures closely approaching theoretical maximum flame temperature for the fuel in question. Hence, there is an enormous energy change at the point where the burned gases issue from the burner into the water. This change serves to divide the burned gases into countless increment of bubbles, whose resultant total surface makes for ideal maximum carbon dioxide absorption.

It is apparent that advantages of this system over conventional methods of recarbonation are many. They include controllability, simplicity of equipment, greater chemical efficiency, trouble-free operation, reduced maintenance, elimination of diffuser and corrosion troubles, all at unit costs equal to, or less than, present costs.

It is believed that the water treating engineer now has available a recarbonation unit that entirely eliminates the many disadvantages inherent in the now conventional equipment. This new tool should be carefully considered for both existing lime softening installations and contemplated ones. It is suited to installation and operation in existing plants without interruption of plant operation, and carries advantages such that its use warrants consideration.

A Study of Procedures for Detection of Coliform Organisms in Minnesota Drinking Water

By Dean M. Taylor

CERTAIN selective media have been proposed for the detection of the presence of the coliform group of organisms in water. A few solid media have been used but most of them are liquid.

Lactose broth has been the standard primary medium for many years in the United States and Canada and the last (eighth) edition of *Standard Methods of Water Analysis* (1) specifies its use for detection of coli-aerogenes organisms in water. Many selective liquid media have been proposed as primary media but they have never been universally accepted by bacteriologists. A study was undertaken by the Standard Methods Committee under the direction of Ruchhoft in 1934–35 with the cooperation of laboratories widely distributed over the United States and Canada (2). Certain selective liquid media were used but they were not considered satisfactory for use as primary isolation media in place of lactose broth because of their under-productivity with strains of coliform isolated from water. Brilliant green bile broth, fuchsin lactose broth, crystal violet broth and formate ricinoleate broth were found to be the most inhibitive to spore-bearing lactose fermenters and their use was suggested in the confirmation procedure. The eighth edition (1936) of *Standard Methods* permits the optional use of these four liquid selective media in both the confirmed and completed tests. All are equivalent for the purpose of these tests but it is recommended that the laboratory worker support his choice of material used by statistical evidence.

The study of these selective media proposed for the detection of the presence of coliform organisms in water was continued in 1936–37 by the Committee on Standard Methods under the direction of

A paper contributed by Dean M. Taylor, Assistant Public Health Engineer, State Board of Health, Minneapolis.

McCrady in an attempt to determine the comparative practical utility of these media when employed in routine water analysis (3). The results indicated not only that the brilliant green bile confirmatory method was the most generally satisfactory of the selective procedures but also that it compared very favorably with the usual *Standard Methods* "completed test." The results obtained from finished waters suggested the use with such waters of the alternative "completed test" of *Standard Methods.*

This present study was carried out with two main purposes in mind:

1. To decrease the time required in testing samples of drinking water for the presence of coliform organisms.
2. To eliminate, if possible, the time and material wasted in examining atypical colonies on Endo or E.M.B. agar plates which are not of the coliform group.

Brilliant green lactose bile and fuchsin lactose broth were the only liquid selective media used because of the limited amount of work which could be done.

Outline of the Work

Difco dehydrated media were used, their composition and the methods of sterilization being according to *Standard Methods.*

Routine drinking water samples were examined for the presence of coliform organisms. Samples of 100 ml. were collected in sterile glass-stoppered bottles which contained 20 ml. of quadruple-strength lactose broth and inverted gas tubes. The bottles were brought to the laboratory, incubated at 37°C., examined at the end of 24 and 48 hours and the presumptives confirmed by the "completed test." In addition, a loopful of culture was transferred from each lactose broth presumptive to brilliant green lactose bile and fuchsin lactose broth and the production of gas in these media followed by the "completed test."

The summary of the results will be given in two parts: first, a comparison of the three methods of confirmation of the lactose broth presumptives from the standpoint of both the aggregate and individual results; and second, a comparison of the relative numbers of typical and atypical colonies on E.M.B. agar and the results obtained by the "completed test" from the respective agar plates.

The three confirmatory procedures (aggregate results) include the usual "completed test," using E.M.B. agar and the two "completed

tests" of *Standard Methods* (1936) using brilliant green lactose bile and fuchsin lactose broth.

Lactose broth presumptives were confirmed by the three methods and the aggregate results are given in Table 1. The figures in column 1 show that more confirmatory presumptives were obtained with fuchsin lactose broth than with brilliant green lactose bile, whereas the proportion of the presumptives which yielded coliform isolations was greater when brilliant green lactose bile was used, as shown in column 2.

The figures in column 3 represent the number of lactose broth tubes from which coliforms were not isolated by the particular procedure

TABLE 1

Results of Three Different Methods of Confirmation of 1,534 Lactose Broth Presumptives by "Completed Test" Method

1		2		3	
CONFIRMATION PROCEDURE	PRESUMPTIVES	COLIFORMS ISOLATED BY "COMPLETED TEST"		COLIFORMS ISOLATED FROM CORRESPONDING LACTOSE BROTH PRESUMPTIVES BY OTHER PROCEDURES	
		No.	Per Cent of Presumptives	No.	Per Cent of Presumptives
SMCT*	1534	701	45.7	75	4.9
BG+SMCT	737	664	90.2	104	14.1
Fu+SMCT	750	668	89.2	103	13.7

* The symbol SMCT signifies the Standard Methods "completed test," and symbols BG and Fu signify the use of brilliant green lactose bile and fuchsin lactose broth, respectively, as intermediate confirmatory media.

in question but from which they were isolated by one or more of the other methods. The total number of lactose broth primary tubes from which coliform organisms were isolated by one or more methods was 776. The usual "completed test" failed to detect the presence of coliform organisms in 4.9 per cent of the presumptives or 9.7 per cent of all presumptives proved to contain these organisms.

The "completed test" results secured through the use of all the confirmatory methods used in the present study are used as a performance standard rather than the usual "completed test" results. The figures in Table 2 represent the percentages of the number of sample portions from which coliform organisms were isolated by any of the three confirmatory methods. The usual "completed

test" using E.M.B. agar is considered as one confirmatory procedure.

It will be observed in Table 2 that the usual "completed test" detected the presence of coliforms in only 90.3 per cent of the sample portions which were proved to contain them. On the other hand brilliant green lactose bile indicated the presence of the group in 95.0 per cent and fuchsin lactose broth in 96.7 per cent of the number of sample portions known to contain coliform organisms. The last column shows that the usual "completed test" yielded more coliform isolations than brilliant green lactose bile or fuchsin lactose broth.

The three confirmatory procedures (individual results) are compared to avoid the counterbalancing of negative and positive errors which occurs in the compilation of aggregates. All positive differences are summed up to give a total called the positive cumulative error and all negative differences are likewise summed up to give the negative cumulative error. For example, whenever a brilliant green lactose bile presumptive occurs and coliform organisms are not isolated by any of the confirmatory procedures the result is recorded as a positive cumulative error. All such results are added together to give the positive cumulative error of the brilliant green lactose bile method. Likewise, whenever the brilliant green lactose bile procedure fails to produce a presumptive and coliforms are isolated by any of the confirmatory procedures, the result is recorded as a negative cumulative error.

TABLE 2

Results of the Three Methods of Confirmation of 1,534 Lactose Broth Presumptives in Percentages of Total of 776 Sample Portions from Which Coliform Organisms Were Isolated by One or More of the Three Methods

PROCEDURE	CONFIRMATIONS PER CENT OF 776	COLIFORM ORGANISMS ISOLATED PER CENT OF 776
SMCT	90.3	90.3
BG	95.0	85.7
Fu	96.7	86.3

Table 3 shows the cumulative errors of the results obtained by each confirmatory method from the 1,534 lactose broth presumptives. Table 3 shows that the negative cumulative error for the *Standard Methods* "completed test" was greater than that for brilliant green

lactose bile and less than that for fuchsin lactose broth. The positive cumulative error for the brilliant green lactose bile method was only 4.63 per cent, whereas, it was 7.99 per cent for the fuchsin lactose broth method. There can be no positive cumulative error

TABLE 3

Cumulative Errors of Results From the Three Confirmatory Procedures (Comparing the Usual Standard Methods "Completed Test" with Brilliant Green Lactose Bile and Fuchsin Lactose Broth Confirmatory Procedures. Presence of Gas Alone is Considered to Constitute Confirmation)

PROCEDURE	CONFIRMATIONS			
	Positive		Negative	
	No.	%	No.	%
SMCT			75	9.66
BG	36	4.63	73	9.41
Fu	62	7.99	84	10.83

TABLE 4

Comparison of Types of Colonies Growing on Eosin-Methylene Blue Agar Used in the Usual "Completed Test" Procedure

Total No. of Agar Plates	1,534
No. of Plates Showing Typical Coliform Colonies	380
No. of Plates Showing Typical Coliform Colonies Which, When Examined by the Usual "Completed Test," Showed the Presence of Coliform Organisms	361
No. of Plates Showing Atypical Colonies	787
No. of Plates Showing Atypical Colonies Which, When Examined by the Usual "Completed Test," Showed the Presence of Coliform Organisms	317
No. of Atypical Colonies Which, When Examined by the Usual "Completed Test" Did Not Show the Presence of Coliform Organisms	470
No. of Plates Showing No Growth	367
Total No. of Plates from Which Coliform Organisms Were Isolated	678
Total No. of Plates Showing Growth	1,167

for the usual "completed test" because it does not record non-confirming lactose broth presumptives as positives.

The positive and negative cumulative errors of any presumptive confirmatory procedure must be very small compared with the total number of sample portions shown to contain coliform organisms,

and from this standpoint brilliant green lactose bile gives better results than fuchsin lactose broth.

The eosin-methylene blue agar plates used in the usual "completed test" procedure were examined for typical and atypical colonies and the results are given in Table 4. Seven hundred eighty-seven or 67.4 per cent of the plates showing growth had atypical colonies. Coliform organisms were isolated from only 317 or 40.3 per cent of these plates showing atypical colonies. That means that 470 or 59.7 per cent of the plates showing atypical colonies had to be examined further but coliform organisms were not isolated from them. In fact, coliform organisms could not be isolated from 42 per cent of the plates showing any growth. The time and materials consumed in making further examinations on the latter were just wasted.

It is a well known fact that chance and the personal factor are involved in fishing colonies from Endo or E.M.B. plates. Typical colonies are easily recognized but the atypical colonies present a difficult problem. Many atypical colonies are encountered when examining drinking water supplies, both finished and unfinished, for coliform organisms. They must be completely confirmed by the "completed test" and much time and materials are wasted in examining these colonies because approximately 60 per cent of them are not of the coliform group of organisms.

The aggregate results show that both the brilliant green lactose bile and fuchsin lactose broth confirmatory procedures yielded results that were closer to the truth than were the results of the "completed test" procedure. If isolations of coliform are required, the brilliant green lactose bile and fuchsin lactose broth methods yield almost as many coliform as the "completed test" method. On the basis of all isolations secured, brilliant green lactose bile yielded 85.7 per cent, fuchsin lactose broth 86.3 per cent, and the usual "completed test" 90.3 per cent of the total number of coliforms isolated.

From a standpoint of individual results the brilliant green lactose bile confirmatory procedure has the smallest negative cumulative error and the fuchsin lactose broth confirmatory procedure the greatest. The brilliant green lactose bile procedure has a positive cumulative error of only 4.63 per cent, whereas, the same error for the fuchsin lactose broth is 7.99 per cent.

The total cumulative error for the brilliant green lactose bile procedure is 14 per cent and 18.8 per cent for the fuchsin lactose broth

procedure. The "completed test" procedure has no positive cumulative error so its total cumulative error is 9.66 per cent. However, the positive cumulative error of 4.3 per cent for the brilliant green lactose bile confirmatory procedure as compared to the zero positive cumulative error for the usual "completed test" seems permissible when considering all the time and materials used in getting the respective results.

Summary

One thousand, five hundred and thirty-four samples of drinking water from ground and surface water supplies in Minnesota and from which lactose broth presumptives were obtained were examined by testing 100 ml. portions of the samples. Three different methods of confirmation of these presumptives were employed: the "completed test" of *Standard Methods* and the two selective liquid media methods, employing brilliant green lactose bile and fuchsin lactose broth.

The E.M.B. plates used in making the usual "completed test" were examined for the presence of typical and atypical colonies. Sixty-seven per cent of the colonies were atypical and only 40 per cent of these atypical colonies yielded coliform organisms when completely confirmed by the "completed test."

The results indicated the brilliant green lactose bile confirmatory method to be somewhat better than the fuchsin lactose broth confirmatory method. The results also indicated that the brilliant green lactose bile method compared very favorably with the usual "completed test" of *Standard Methods* and might advantageously replace it. When it was desirable to attempt to isolate coliform organisms, the alternative "completed test" employing brilliant green lactose bile was preferable to the usual "completed test."

The author wishes to express his appreciation to M. H. McCrady for his helpful criticism and advice.

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Swimming Pool Water Treatment

By Herbert L. White

ABOUT forty years ago, the only places available for swimming were the old swimming holes in the creeks or the sand beaches along the rivers and lakes. I can remember when I was a boy that every once in a while we would hear about boys or adults who had drowned while swimming in the river. Most of the accidents were caused by divers hitting submerged stumps, rocks, etc., and by weak swimmers wading into concealed deep holes called "step-offs."

As a result of these accidents, artificial pools were built to remove the hazard of "step-offs," hidden stumps, and undertow. These pools were called fill-and-draw pools, that is, they were filled and then drained at the end of some definite period, such as a week or a month. The water in these pools was quite turbid and concealed disabled bathers, hence there continued to be a few deaths, although not as many as before. The fill-and-draw pools, however, had one distinct disadvantage, for they became a very good channel for spreading diseases. Hundreds of persons used the same water in the pool before it was changed, and as each person went into the pool, he carried in his quota of germs and body dirt. Consequently, infections developed easily, particularly ear infections. An attempt was made to alleviate the condition by adding fresh water daily, but this was not satisfactory.

Then someone suggested that the difficulties caused by the cloudy water and by the multitude of germs in the water could be corrected by means of a system which would continuously purify the water. This system consisted of a pump which drew water from one point in the pool, forced it through filters to remove the dirt, sterilized it with chlorine or other germicide, and then returned it to the pool at

A paper presented on April 21, 1939, at the Illinois Section Meeting at Urbana, Ill., by Herbert L. White, Sanitary Engineer, Physical Plant Department, University of Illinois, Urbana, Ill.

another point. This system is called a recirculation system, and was made more elaborate as time went on by adding a heater to heat the water, and by drawing water from the pool and returning it to the pool through many openings.

Many pools are now supplied with such equipment, that is, with pumps, filters, sterilizers, and heaters, but are poorly operated—probably because both operators and patrons are comparing the pools with the old swimming holes, rather than comparing the pools with the perfect pools they could and should have.

Availability of Literature

In comparison with other public health problems, there has been little technical literature available on the operation of swimming pools. Many of the articles which have been available have been influenced by a desire to sell some particular product for use in connection with the swimming pools, and consequently much of such literature is not as reliable as could be desired.

A committee of the Conference of State Sanitary Engineers and of the American Public Health Association prepared a very careful report on the standards which should be used in connection with the design and operation of a swimming pool. This was of considerable value to those who were familiar with it, but unfortunately it did not fall into the hands of many operators of small pools. Finally, in 1931, the State Department of Public Health was charged with the responsibility for the sanitation of swimming pools in Illinois. The department has improved the situation materially through swimming pool sanitarians who inspect the pools, through checking plans for all new swimming pools, and through various literature which they prepare and distribute to interested parties. However, if we can judge the situation in the state by the pools with which we are familiar through contacts with various persons who have visited our pools or asked us for information regarding the operation of the pools, there is still much improvement to be made.

The operation of swimming pools is of particular interest to water works men because the equipment used is very similar to that used in water treatment, and because swimming pools are a public health problem very similar to that of a public water supply. For the benefit of members of this section who are actively interested in the operation of pools, the writer will relate some of the difficulties experienced at various pools.

The Problem of Turbidity

One of the most baffling problems to the average swimming pool operator with whom we have talked is the problem of turbidity. It is likewise one of the most important problems, for the condition of the pool is frequently judged by the appearance of the water. Many persons do not realize that the water for a swimming pool must be much clearer than that which is satisfactory for drinking water. The reason for this is that drinking water is required only to be clear enough so as not to be objectionable in a glass of water, whereas, in a swimming pool the water must be clear enough to permit a disc to be seen on the bottom of the pool under six or eight feet of water. Water which has such an extremely low turbidity is beyond the range of an ordinary turbidimeter and must be examined by means of an ultra-turbidimeter, similar to the St. Louis or Baylis type.

Turbidity is caused by algae, chemicals, dirt carried in by bathers, dirt blown in, or dirt entering with fresh water in excess of that removed by the purification equipment. Usually when the turbidity of the pool becomes unsatisfactory, the operator spends his time wondering why the filters are not functioning properly, whereas frequently the trouble is due to the excessive amount of dirt carried into the water by bathers. Few people appreciate how greatly the latter can affect the condition of the pool. For instance, one year there were two groups of young boys who had a swim in a certain pool. The first group had soap, and very carefully took hot shower baths. After their swim the pool was almost as clear as it was before. The second group, however, did not bring soap but attempted to take shower baths without it. As a result, the water was quite turbid after they left the pool and it was necessary to close the pool for a few days to permit the filters to remove the dirt. Similar instances have been repeated many times and are evidence that the swimming pool may serve as a public bathtub unless bathers take a cleansing shower, consisting of soap and hot water, before entering the pool. If one stops to think, when he sees a cloudy pool, he will realize that the turbidity is due to foreign matter in the water, and this foreign matter is probably dirt carried in on the bodies of bathers.

Of course, we might buy bigger pumps, filters, and sterilizers, and not try to reduce the pollution carried in by bathers. However, very few of you would want the sewage of a community dumped

into the stream just above your intake, regardless of the size of your treatment plant. Since the operator thinks first of the filters when high turbidity occurs in a pool, the natural move for him to make is to add more alum, and if it does not work, then he adds copper sulfate, hoping to kill algae if they are responsible for his trouble. The alum frequently goes through the filters in solution and forms a floc in the pool to add to the turbidity. The writer knows of one pool where thousands of dollars were spent in an effort to obtain a clear water. The solution was simple enough as was discovered

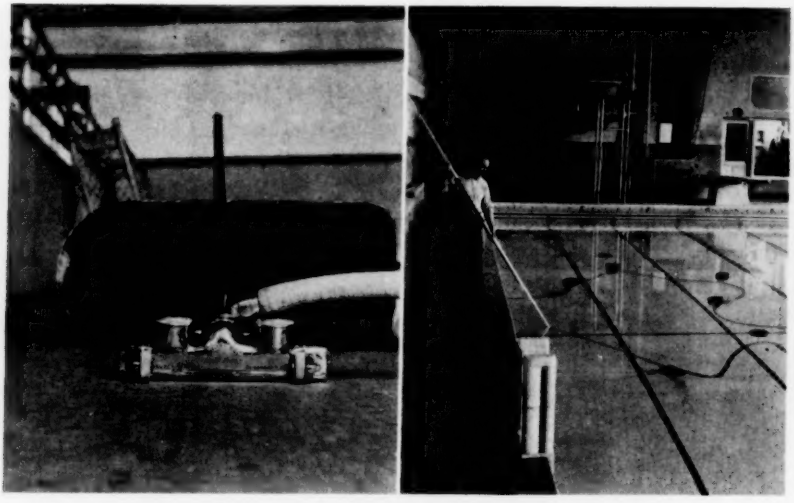


FIG. 1. Vacuum Cleaner Used for Cleaning Bottom of Pool. Note that nozzle is only ¼ in. from floor.

FIG. 2. Cleaning Pool with Vacuum Cleaner. The suction hose is supported with floats.

later, because the trouble was due to just two things: the excessive amount of dirt carried in on the bodies of bathers, and the excessive amount of alum added to the water. The operators kept building bigger and bigger alum pots, and finally had one pot large enough to hold one hundred pounds of alum. This same general problem has existed in other pools with the additional factor that, where chlorinators are not available, turbidity is constantly added to the water in the form of milky chloride of lime solutions. This places an unnecessary load on the filters and frequently is sufficient by itself to raise the turbidity to a point where the pool is unsafe.

Operators lose sight of the fact that a recirculation type of pool is just like a steam boiler. If all the steam from a boiler is condensed and returned to the boiler as feed water, the boiler remains in good condition, and the chemicals dissolved in the boiler water are small in amount. On the other hand, if the condensed steam, before it is returned to the boiler, has large quantities of chemicals added to it each time, it will not be long until there will be an enormous quantity of chemicals in the boiler and boiler water. With a swimming pool, if chemicals added to the water are restricted to a small amount of alum and chlorine gas, then the water will contain little mineral matter. However, if alum, copper sulfate, pH bricks, and calcium hypochlorite are poured in freely, the mineral content of the water will build up and will have a tendency to form deposits in the recirculation equipment, and the pool will contain a turbid water.

A water works man will need no instruction if the purification system alone is at fault, because it is not a difficult matter for him to check the loss of head through the filters, the condition of the filter sand, the position of the valves, and the condition of the pump, to make certain water is being filtered at the proper rates. Pressure filters are usually operated at two, three or four gallons per square foot of sand surface per minute. It is possible, however, that the differential pressure gage and the elbow meter (Figs. 3 and 4) may be of value in the above work. The latter, particularly, is a very desirable instrument, and it does not cause a pressure loss nor does it serve as a trap for sand, etc., as an orifice or a water meter would do. Furthermore, the elbow meter can be calibrated in place quite easily by running water through it at a few known rates of flow.

Algae in the Swimming Pool

One spring, algae were noticed growing on the sides of a pool. Copper sulfate was tried but was not effective. Samples were scraped off the walls and taken to a botanist who said the walls would have to be scrubbed before copper sulfate treatment would be effective. This was done and 20 p.p.m. copper sulfate was added to the pool. The algae, however, continued to thrive. They were killed a little later by over-chlorinating the pool. The residual chlorine was raised to about 5 or 10 p.p.m. This had the advantage that not only was the water in the pool sterilized, but so also was the entire system, that is, the circulation lines, filters, etc

Importance of Color

Many patrons of swimming pools desire a blue water. It has been noted that pools in central Illinois, where the water contains some iron, are sometimes green and sometimes blue. A demand for a blue water sometimes has been so strong that the writer has threatened to color it with a blue dye. One operator, James Quick, noticed that water from the general water supply, which has a dark grass-green color, is changed by over-chlorination to a beautiful blue. It

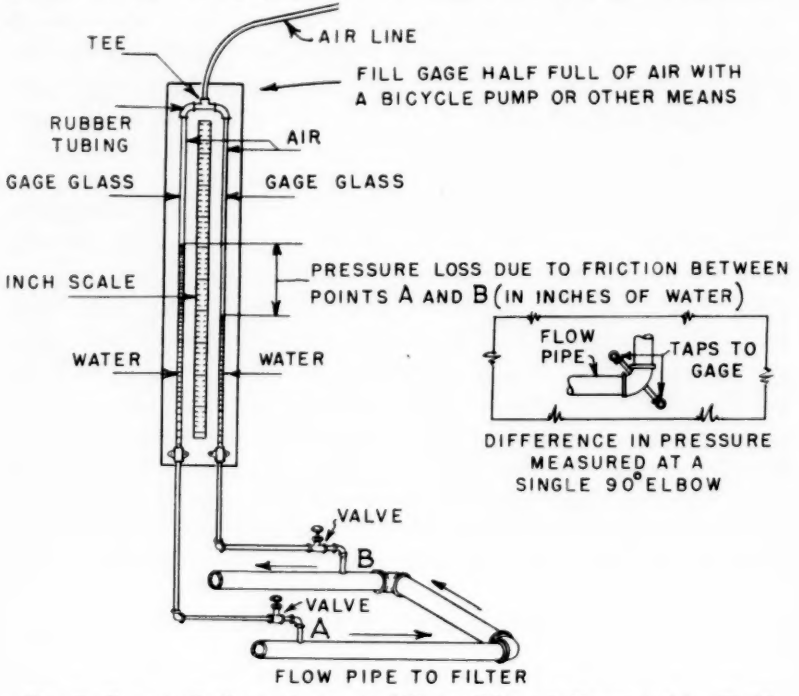


FIG. 3. Gage to Indicate Amount of Water Filtered (Suggested by W. M Lansford).

is accompanied by the deposit of a rusty precipitate on the bottom of the pool, which the author has assumed is iron or iron organisms which have been thrown out of solution or suspension. This pro cedure is also of value in that it reduces the turbidity of the pool somewhat.

For several years the author determined pH by comparing the colors developed by standard dyes with a printed color chart. The

results were consistently high. Finally, a set of sealed liquid color standards was purchased which has been very satisfactory.

The department's chlorinators are operated entirely with pool water. This is done because the pool water is clearer than tap water, hence the chlorinator requires less cleaning. Another advantage is that the temperature of the pool water is warmer and there is less likelihood of liquid chlorine damaging chlorinator parts. Moreover, some chlorinators, if placed below pool level, may serve as a cross-connection. It is true that the connection is guarded by chlorine

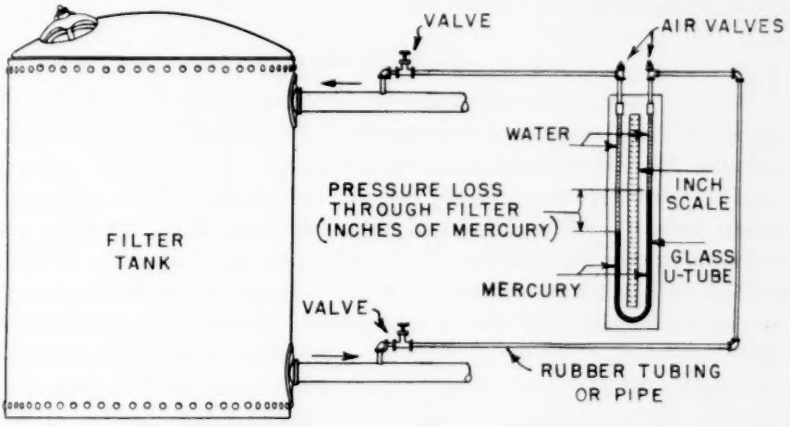


FIG. 4. Gage to Indicate Pressure Loss through Filter. Note: A gage of the type shown in Fig. 3 can be used for this purpose if the filter pressure loss is not so great that a water column gage would be too large.

most of the time, but the author prefers to remove the connection and avoid speculation.

Temperature

The A. P. H. A. standard calls for a water temperature not to exceed 78°F., but the women have never been convinced that 78 degrees is warm enough. They wish to have the water temperature in the pool not less than 80. The men's pools are maintained at 78, however.

Water in this locality has an alkalinity of more than 300 p.p.m. When the water is used in a swimming pool, the chemical, bacterial, and physical qualities improve as time goes on. For this reason, we have connected the drains from the overflow section (scum gutter)

to the recirculation system to save the water normally lost by wave action. It makes unnecessary the addition of large amounts of raw water which would introduce calcium to clog the heater and filter underdrains, and introduce small amounts of iron to give the pool water an unattractive color and increase the turbidity. This method is called a closed system, and we believe it will be widely used, particularly in areas where the chemical quality of the water is unsatisfactory. On more than one occasion, we have been unable to keep a pool water bright and clear merely because we were adding too much fresh water.

Chlorination

One fall, just before school was opened and the chlorinator started up, when a pool was completely filled with raw water having a residual chlorine content of approximately 1.0 p.p.m., it was found after a day or two that the color developed with ortho-tolidine indicated about 3 p.p.m. This did not seem to be improbable, but after the chlorinator was shut down, the chlorine did not drop as it should. Finally, it dawned on us that we were obtaining a false color due to the presence of nitrites which had formed in the filter sand while the filters were out of service. Since then we have followed the practice of using an ortho-tolidine solution containing excess HCl, as suggested on page 1662 of a paper by Tarvin, Todd and Buswell (Jour. A. W. W. A., **26:** 1645), which will prevent any appreciable error if read at the end of five minutes. If no chlorine is present and if nitrites are present, the color developed is a decided off-shade, which immediately indicates to the attendant that the color is a false one. Afterward, a procedure for reducing nitrite color was published in *Standard Methods of Water Analysis* (1936). This, however, calls for two solutions to be used, one a standard ortho-tolidine solution, and the other an acid solution. The acid solution is used, of course, when it is thought nitrites might be present. It has seemed to us to be much safer and more practical to use the solution suggested by Buswell, which is satisfactory either in the presence or absence of nitrites, rather than to tell an attendant to add the acid solution, *if nitrites are present.* Nitrites may be present only on rare occasions, just as they were with us, and the attendant would be caught unaware.

It has been difficult sometimes to maintain the chlorine content of the pool within the 0.3 to 0.6 p.p.m. limits, and for that reason, we keep on hand some calcium hypochlorite and a solution of sodium

thiosulfate, with which we can without delay either raise the chlorine content if it is much below the limit, or reduce it if it far exceeds the limit given. We feel that the chlorine content can be much higher than the 0.6 p.p.m. without harmful effects to the bathers' eyes, because on one or two occasions, students have entered a pool when we were not prepared for them and when the chlorine content was in the neighborhood of 2 p.p.m. No unpleasant after-effects were reported.

One sanitarian told us that ammonia is constantly introduced into swimming pools by bathers through urine and perspiration. If this is true, possibly we never have chlorine in the pool water, but have chloramine, and should maintain a residual chlorine content of from 0.5 to 1.0 p.p.m.

One thing for which we have no answer is the problem of eye irritation to bathers. As mentioned above, we have seen bathers after using a pool with 2 p.p.m. residual chlorine with no bad results. On the other hand, we have had reports of eye irritation in pool water which had a pH of about 7.6, an alkalinity of 200 p.p.m., and a chlorine content of 0.5 p.p.m. Another pool, which seemed to us to be exactly similar in every respect, was perfectly satisfactory to the same bathers.

Education

The Illinois State Department of Public Health has been active during the last few years in attempting to educate the operators, to assist them in producing a safe sanitary water, and also to convey to them the responsibility which they should bear. This has not been an easy task. To illustrate this point, we recall a meeting of swimming pool operators where it was mentioned that algae could be controlled by over-chlorination, yet one of the operators, who was there at the time, later had trouble with algae and did not know how to take care of the situation.

The Health Department is attempting to raise the level of pool sanitation by operators' meetings, by a magazine for swimming pool operators, and by inspection and grading of pools. These all have considerable merit. The weights of some of the various factors in composing a grade might well be shifted somewhat, because, in our experience, we have found that the operation is just as important as the equipment, and we have noted pools which have excellent equipment which could not produce a clear water. We have also

noted pools which have very poor equipment which have produced quite satisfactory water due to good operation. We believe that pools in the latter class should not be penalized unduly because, after all, they have obtained the end desired, namely, a safe, sanitary water.

It will also be necessary to educate the public, or the patrons, as to what their responsibilities are in connection with sanitation of the swimming pool. For instance, we often hear the slogan "Swim in drinking water." However, at two pools we know of, feces were found in the water, and at another, a girl dropped a sanitary napkin just prior to entering the pool. Surely, these patrons did not have any conception of their proper responsibilities to maintain "drinking water standards." It is possibly as important for the patrons to use utmost care and cleanliness to reduce the quantity of filth and germs carried into the water as it is for the operator to remove these.

Grounding of Electric Circuits on Water Pipes

By J. B. Downer

THE title limits this discussion to intentional connections between electrical circuits and water pipes.

Let us further limit it by excluding from consideration any effect of direct current and agreeing that adequate grounding of certain electric circuits is necessary for the protection of life and property.

The application of a particular scheme is largely a local problem, because it is affected by so many variables. Of these variables, technical considerations are becoming second to tactics, in importance to water works managers.

Separate groups, organized on a national scale, have united upon a program, expressed in part, in the National Electrical Code, and numerous state and local laws, which will relieve the electric utilities of legal responsibility for faulty grounds; will relieve them of much of the financial burden of providing and maintaining ground protection; will tend to lower the standards of construction both for interior wiring systems and electrical distribution systems; and at the same time will eliminate the electrical utility as a party to any legal contract to protect the water works system.

The author once thought that A. W. W. A. representation on the National Electrical Code Committee would result in a code which would secure adequate protection to electricity consumers, by permitting restricted, controlled, and intelligent use of piping systems wherever consistent with good practice, without jeopardizing the interest of water works.

The present situation proves that one vote on a subcommittee is useless when an opposing interest is represented by multiple organizations and enjoys the passive acquiescence of other groups.

A paper presented on May 11, 1939, at the Pacific Northwest Section at Tacoma, by J. B. Downer, Engineer, Water Department, Seattle, Washington.

As the author sees it now, the independence of water works can be safeguarded only by opposing any grounding scheme (by legislative lobbies, legal action, and publicity) unless such scheme has been approved by a committee of the A. W. W. A. Grounding on consumers' premises would not survive one session of the legislature if the public really understood its implications.

If a power company is actuated only by a desire to protect its customers, it will readily guarantee that contacts on water pipes shall conform to the following requirements:

(1) No pipe shall be substituted for a circuit conductor (Figs. 1 and 2).

(2) No pipe shall serve as the only path to earth for current from any circuit (Figs. 3, 4 and 5).

(3) The grounded conductor shall be of adequate size to insure its continuity under any *probable* circumstance which would not disrupt the other wires of the circuit (Fig. 5).

(4) The grounded conductor shall be of such size, relative to the other circuit wires, that current in the pipe will not be objectionable (Fig. 2).

(5) The right and *power* to remove any ground contact shall be secured to the water works manager (Fig. 6).

These conditions can not be met by any grounding scheme which includes grounding circuits on the water service on the consumers' premises.

As long as ground contacts are confined to neutral mains and/or transformers, the water works manager can make a binding agreement requiring the electric utility to maintain adequate conductance between grounds and to remove objectionable grounds. Once electric services have been legally grounded to water pipes on consumers' premises, the water works management has lost control of the situation. The man who made the contact was obeying the law and can not be held for any consequences; the owner did not even know that the contact existed; the electric utility (the beneficiary) is entirely out of the picture; the wiring inspector has the authority and it is his duty to prevent any interference with the ground. In short, the water piping system is under multiple management. (The scheme is still a little weak. It does not prohibit the use of non-conducting material for pipes or joints. That may come later.)

Where a water works system and an electrical distribution system supply the same area and are under the same ownership, they are

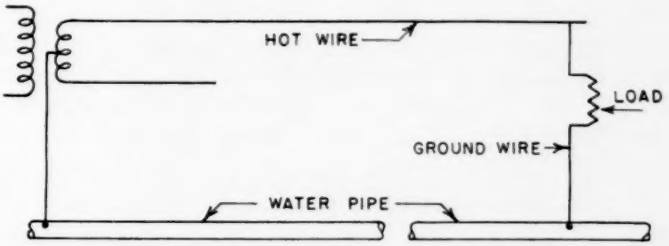


FIG. 1. Water Pipe Substituted for Neutral Conductor. Service to electricity consumer is normal, if the water pipe is an effective ground. Current in pipe is equal to current in hot wire. Even at a corrosion efficiency of 1 or 2 per cent, this scheme would endanger the pipe. The water pipe is not always electrically continuous.

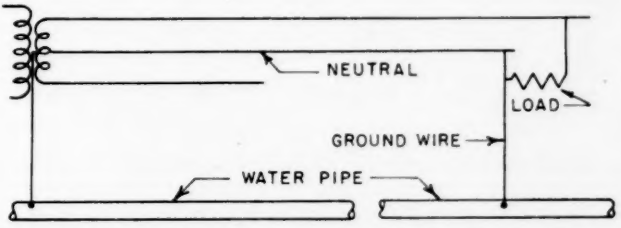


FIG. 2. Neutral in Parallel with Ground. Unbalanced load divides between ground and neutral in proportion to their conductances. If neutral is maintained of adequate size, current in ground will be held to tolerable magnitude.

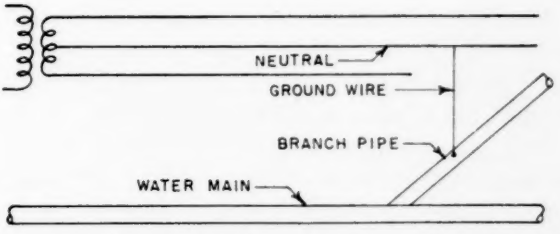


FIG. 3. A Man opening the branchpipe between the main and the ground connection would be subject to the full potential between the neutral and the earth. With a high-potential high-resistance cross on the neutral, he might be subjected to the full potential between the high voltage line and earth.

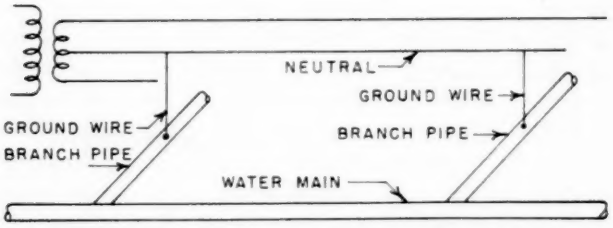


FIG. 4. Multiple Grounds on Neutral. Man opening either branch would be subject only to the potential drop across the other ground and that in the neutral. If the neutral is of adequate size and the distance between grounds is not excessive, the water works employee would not be endangered.

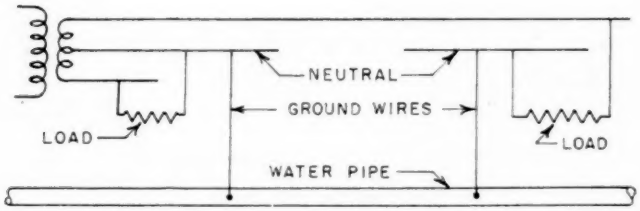


FIG. 5. Neutral Broken Between Grounds (possible by sleet, e.g.). Load on outside wires balanced by current through ground. If pipe extending beyond ground contacts does not constitute an effective ground, a man opening the pipe between grounds would be subject to full potential between circuit and earth. Adequate neutral *plus* multiple grounds are essential for reliable protection to water works employees.

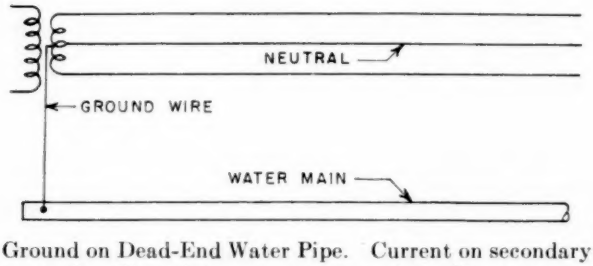


FIG. 6. Ground on Dead-End Water Pipe. Current on secondary has only one path to earth. Hazard to man opening pipe. Power to remove this ground is power to secure a second path.

almost invariably interconnected. This is logical because the advantages to the electrical system far outweigh the disadvantages to the water system, and the common owner can well afford to suffer the disadvantages in order to gain the advantages.

Where the ownership of the utilities is separate, *all* of the advantages of such interconnection accrue to the electrical system and *all* of the disadvantages are borne by the water system.

The water works has no duty or moral obligation to permit the use of water pipe grounds. Acquiescence in such a program must rest entirely upon the protection to life and property, offered by the scheme proposed. There is no incentive whatever to provide or supplement conductance for commercial circuits.

Grounding circuits upon consumers' premises provides little, if any, more protection than can be secured by adequate grounds on the neutral main. It is debatable whether it may not encourage construction methods which result in less protection.

An engineer designing electrical distribution systems is under the same stress to reduce costs as is an engineer designing water systems. With all services grounded on consumers' premises, he will be under enormous temptation to minimize or eliminate neutral copper. His customer would never know the difference in localities where the water pipe does in fact provide an adequate ground (Fig. 1). Without grounds on consumers' premises, you would hear very little discussion of bare neutral.

We have heard lately of cases where alternating current is assumed to be responsible for corrosion, bad tastes, etc. We are beginning to hear the claim that certain household or industrial devices tend to rectify alternating current and thus cause corrosion. Some suspect that corrosion may occur during the positive half cycle and that the metal ion may form an insoluble compound with some component of the electrolyte, so that it can not be plated back during the negative half cycle. This might cause corrosion, at each electrode, equivalent to one-half that caused by a corresponding direct current. Oxygen dissolved in the electrolyte might form the insoluble compound with the metal ion, but it apparently does not react in the one one-hundred-twentieth of a second allowed by sixty-cycle current. In the few reports which the author has seen, no mention has been made of the means used to determine that direct current or galvanic action was not a factor in the corrosion. The

Bureau of Standards made extensive tests on lead and iron.* The Bureau found corrosion on iron in soil to be negligible at sixty cycles, reaching a corrosion efficiency of 1 or 2 per cent in soil to which sodium carbonate had been added.

The A. W. W. A. has competent representation on the committee now investigating alternating current corrosion. Pending the committee's final report, the author would not say that alternating current corrosion is negligible under existing grounding rules. The author does not believe, however, that it is a major consideration, if the conditions outlined above are met.

* Technologic Paper No. 72.

Filter Sand

By Thomas M. Riddick

THE original work on filter sand was done by the late Allen Hazen about 1890. Since that time much excellent research has been contributed. Hazen introduced the terms "effective size" and "uniformity coefficient" which are in use today. As Baylis has pointed out, however, Hazen's work was primarily applicable to "slow" rather than to "rapid" sand filters.

Sand employed in the former was often unwashed and ungraded and hence contained a relatively high per cent of "fines" and the sizes extended over a wide range. It was not uncommon to have an effective size as low as 0.2 mm. with a uniformity coefficient of 3.0 or higher.

With these conditions, Hazen found that the fine sand (0 to 10 per cent) had about as much effect upon filtration efficiency as practically all the other sizes combined. This is natural.

Today, the "fines" (retained on 150, 100 and 65 mesh), as well as the coarse sand (retained on 12 and 14 mesh screens) are eliminated by the producer through washing and screening. Under these circumstances, effective size and uniformity coefficient lose much of their original significance.

There are two screen scales available. The clear opening between meshes of the "square root of 2 series" increases in the ratio of 1.414. This is entirely adequate for filter sand analysis. The greater refinement enabled by the "fourth root of 2 series" (1.189) is not warranted.

According to the theory of hydraulic subsidence, backwashing *should* grade sands within a layer, leaving the coarsest material at the bottom and the finest at the top.

A paper presented on November 2, 1939, at the New York Section Meeting at Utica, N. Y., by Thomas M. Riddick, Consulting Engineer and Chemist, New York.

An examination was made of six filters to determine whether or not stratification *actually* takes place, and this subsequently led to other conclusions with regard to the placement of wash water troughs, desirable size of sand, rate of backwash, etc.

Seven composite samples were taken from the following sections of each filter layer: the top $\frac{1}{2}$ in., $\frac{1}{2}$ in. to 2 in., 2 in. to 6 in., 6 in. to 12 in., 12 in. to 18 in., 18 in. to 24 in., and 24 in. to 30 in.

Filter Sand Requirements

A filter sand should be used which will produce a clear and sparkling effluent—free from coagulant—under the *worst* rather than *average* conditions of floc formation. The poorest coagulation is generally experienced during the fall season.

Filter sand should also *completely* prevent the passage of activated carbon when this is applied to the top of the layer. This important qualification has arisen only within the past few years.

Weak floc formation in the average rapid sand filtration plant is quite comparable to normal conditions in a pressure filter. In the latter, the dosage is often far from optimum and only a few minutes elapse between the application of coagulant and the passage of the treated water through the filter.

Effect of Fine Sand on Clarity of Effluent

Repeated instances have shown that when 35 (and some 48) mesh sand was added to either pressure or gravity filters deficient in these sizes, immediate improvement in the clarity of the effluent was noted. This led to the author's firm conviction that fine sand is essential to the production of a finished water with a turbidity comparable to distilled water.

Although fine sand will undoubtedly give a shorter filter run than coarse, this is partly due to the removal of a greater per cent of suspended matter from the treated water. Long filter runs are more dependent upon the design and efficiency of the mixing basin and the retention period of the sedimentation basin than upon the size of sand employed.

The loss of head through either fine or coarse sand takes place principally at the surface as is shown by the complete recovery of a slow sand filter after skimming.

To obtain a clearer picture of the action of a sand layer during back-washing, let us visualize the particles falling through still water

rather than being suspended by an upward flow, since theoretical and experimental data are available on the former.

Figure 1 shows the rate of fall of sand particles through still water at 10°C. Stokes' law is based upon perfect spheres whereas Hazen's values are the result of experiments on sedimentation with normal (angular) sand grains. It must be borne in mind that both curves apply to the fall of *individual* sand particles in a *large* volume of water, whereas, in a filter, free fall cannot be obtained since the sand grains are crowded and the volume of water is relatively small.

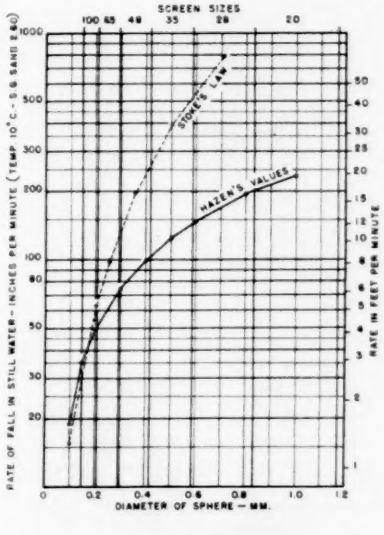

SCREEN SIZES
100 65 48 35 28 20
STOKES' LAW
HAZEN'S VALUES
RATE OF FALL IN STILL WATER - INCHES PER MINUTE (TEMP 10°C - S.G SAND 2.65)
RATE IN FEET PER MINUTE
DIAMETER OF SPHERE - MM.

FIGURE 1

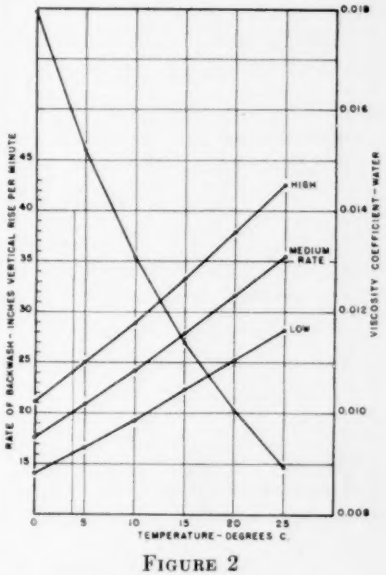


FIGURE 2

The rate of fall of the 65 mesh sand is 5 ft. per min.; of the 48 mesh, 7 ft.; and of the 35 mesh, 11 ft. Torpedo sand (or the top gravel course) falls at a rate of about 30 ft. per min. These velocities are much higher than we normally suppose, and at once we question how 65 and 48 mesh can be lost at a 2-foot rate of backwash.

Figure 3 shows a typical filter with reinforced concrete wash water trough.

Figure 4 presents a hypothetical arrangement of sand or gravel. The horizontal section at *A* represents the top layer of torpedo sand. We find that if the center of each circle is connected by a straight line, for every half circle (represented by the area within the heavy

triangle) there exists a small triangular curved area (shaded). This is only 10 per cent of the total area within the triangle.

Assuming a rate of backwash of 2 ft. per min., based on *total* filter area, from the fundamental equation $Q = AV$, the *vertical* velocity through this *horizontal* plane must be *ten times* this value, or 20 ft. per min.

From Hazen's curve it is shown that to lift torpedo sand a velocity of 30 ft. per min. is required. This then is the reason why torpedo

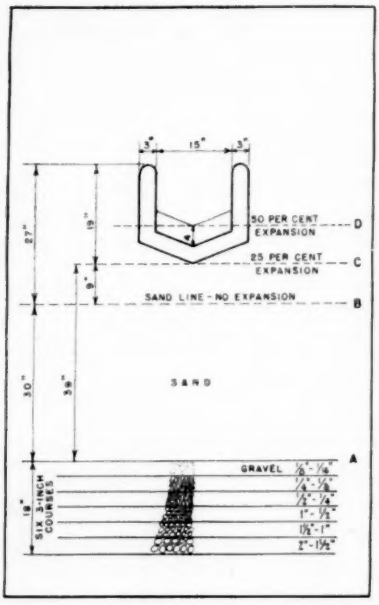


FIG. 3

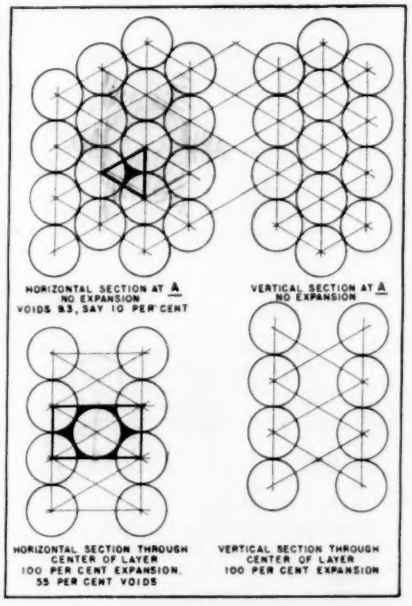

HORIZONTAL SECTION AT A
NO EXPANSION
VOIDS 83, SAY 10 PER CENT
VERTICAL SECTION AT A
NO EXPANSION
HORIZONTAL SECTION THROUGH CENTER OF LAYER
100 PER CENT EXPANSION.
55 PER CENT VOIDS
VERTICAL SECTION THROUGH CENTER OF LAYER
100 PER CENT EXPANSION

FIG. 4

FIG. 3. Typical Filter with Reinforced Concrete Wash Water Trough
FIG. 4. Hypothetical Arrangement of Sand

sand is not expanded during backwashing, which is in accordance with practical observations.

If expansion does not occur, a 2-foot rate would also produce a 20-foot velocity through the sand bed. This rate, however, is capable of lifting all sands 20 mesh and finer. The bed must expand until the effective area through the sand is sufficient to reduce the velocity to a point at which these particles will remain in suspension and will not be carried over into the wash trough.

Let us consider a 100 per cent expansion, as represented by the

drawings in the lower half of Fig. 4. The sketch on the right shows a clear space (approximately equal to one diameter) existing between alternate vertical layers of sand. The effective area in a *horizontal* plane in this instance is indicated by the central circle and the shaded area, in the figure on the left. This amounts to 55 per cent. A 2-foot wash rate with this degree of expansion gives a velocity of 3.6 ft. per min.

At 50 per cent expansion, the effective area will lie between 10 and 55 per cent, say 33 per cent, and the average velocity will be 6.1 ft. per min.

From Hazen's curve, we find that this is sufficient to lift 65 mesh sand and is close to the point at which 48 mesh is raised.

From both theoretical and practical considerations, all 65 mesh sand should be excluded, or, in other words, the "finest" sand which can be maintained in a layer at 50 per cent expansion is the 48 mesh size.

Relation of Height of Wash Water Troughs to Depth of Sand

In Fig. 3, it is observed that a 50 per cent expansion elevates the sand to within the area occupied by the wash water trough. The sudden increase in velocity due to the trough *reducing* the filter area, makes the vertical rise greater than 6.1 ft. per min., and this is sufficient to remove 48 mesh sand.

The values involved are based on theoretical considerations, but it is highly probable that the placement of the wash water trough at an elevation below the line of 50 per cent expansion is largely responsible for the loss of sand, and surely limits the maximum rate of backwash regardless of the capacity of the trough.

The conclusion is obvious—the *bottom* of the wash water trough should be placed above the line of 50 per cent expansion. In this instance, it would require raising the trough a minimum of 7 in. This height should be further increased from 3 to 6 in. to provide a factor of safety.

A 50 per cent expansion is undoubtedly advisable to remove mud balls and maintain sand in good condition.

It will also be noted that the wash water trough shown in the diagram is pitched 4 in. toward the discharge end. In backwashing filters of this type, a lateral movement of water in the direction of the high end is often observed. This is probably due to the pitch,

and suggests that troughs should be built of uniform section throughout, with level bottom.

Had the sand layer been laid to a depth of 22 to 24 in. instead of 30 in., it would have allowed for a 50 per cent expansion without bringing the sand into the constricted area created by the trough.

Elevation of Wash Water Trough

The elevation of the wash water trough should not be determined by the distance from the *top* of the sand to the *lip* of the trough. For a 50 per cent expansion the distance from the top of the layer to the *bottom* of the trough should be at least one-half the depth of the layer.

Wash water troughs constructed of thin walled material such as steel, cast iron, alloys, or transite are preferable to concrete, since the thickness of the latter is responsible for a sizable per cent reduction in the effective area.

Effect of Temperature on Rate of Backwash

It is well known that the rate of backwash must be greater in summer than in winter, and personal observation indicates that the actual expansion of sand at a 30-inch rise in August is somewhat less than that experienced during the winter months at a 24-inch rate.

The major factor is the viscosity of water, which is a function of temperature. Figure 2 gives three curves, based on viscosity, showing the required rate of backwash in inches vertical rise per minute for any temperature.

The data in these instances were 16-, 20- and 24-inch rises at 3.8°C., since all reservoir waters are constant at this value when frozen over. These curves refer to rate of rise based on *total* filter area and not on the *vertical rise* as measured by a hook-gage placed below the lip of the wash water trough. Due to constriction created by the trough, the latter rate is excessive by 25 to 50 per cent. The two bottom curves are more suitable for average plants.

Reservoir waters in southern New York will vary from 3.8°C. to 20°C. For a medium rate of backwash the rise should be from 20 to 32 in. per min. The water of open streams however will vary from 0°C. to about 25°C. The rate of backwash for the same expansion will therefore vary from 18 to 35 in. The maximum temperature of water should be considered in the design of wash water troughs, tanks, valves and piping.

Stratification of Filter Sand

Table 1 gives the results of analyses of samples collected at the Danbury, Conn., plant. The amount of sand retained on the 65 and 14 mesh sieves was less than 1 per cent, which, for practical purposes, may be disregarded. The significant amounts in all filter sand analyses are those retained on the four sieves: 20, 28, 35 and 48 mesh.

This table lists seven individual analyses, each showing "per cent retained" and "per cent passing." From the latter values have been computed the "effective size" and "uniformity coefficient" found in each stratum. Results are presented graphically in Fig. 5.

TABLE 1

Analyses of Sand Filter No. 5, West Lake Plant, Danbury, Conn., August 11, 1939

SIEVE NUMBER	DEPTH IN LAYER																
	0″-½″		½″-2″		2″-6″		6″-12″		12″-18″		18″-24″		24″-28″		Total		
	Per cent		Per cent		Per cent		Per cent		Per cent		Per cent		Per cent		Per cent		
	Ret	Pass	Ret	Pass	Ret	Pass	Ret	Pass	Ret	Pass	Ret	Pass	Ret	Pass	Grams	Ret	Pass
65	1.8	—	1.0	—	2.1	—	0.7	—	0.4	—	0.1	—	0.6	—	6.7	0.9	—
48	80.2	1.8	62.1	1.0	39.1	2.1	20.3	0.7	16.3	0.4	11.9	0.1	9.2	0.6	239.1	34.2	0.9
35	18.0	82.0	36.9	63.1	55.7	41.2	60.2	21.0	61.3	16.7	59.6	12.0	49.0	9.8	340.7	48.7	35.1
28	—	100.0	—	100.0	3.1	96.9	16.3	81.2	19.3	78.0	25.4	71.6	26.1	58.8	90.2	12.9	83.8
20	—	—	—	—	—	100.0	2.5	97.5	2.7	97.3	3.0	97.0	5.2	84.9	13.4	1.9	96.7
14	—	—	—	—	—	—	—	100.0	—	100.0	—	100.0	3.7	90.1	3.7	0.5	98.6
8	—	—	—	—	—	—	—	—	—	—	—	—	6.2	93.8	6.2	0.9	99.1
														100.0	700.0		100.0
Effective Size—mm.		0.35		0.36		0.36		0.39		0.40		0.42		0.42			0.38
Uniformity Coefficient		1.17		1.17		1.36		1.41		1.41		1.39		1.45			1.40

The depth of sand appears vertically from 0 to 28 in. and the per cent of each size of sand found in each stratum may be read at the bottom of the figure.

Thus the top ½ in. contained approximately 80 per cent of 48 mesh and 20 per cent of 35 mesh sand. The effective size for this stratum was 0.35 mm. and the uniformity coefficient 1.17. Most of the stratification took place within the top 2 in., although a definite gradation in both 48 and 28 mesh sizes is also seen from top to bottom. This filter showed the best hydraulic grading of those sampled, though mud balls were present at all depths. The author considers

this an ideal grading, and the presence of the large per cent of 48 mesh on top of the filter gives excellent reduction in turbidity.

Figure 6 shows the results of Mamaroneck, N. Y., samplings. This sand had an effective size of 0.45 mm. and a uniformity coefficient of 1.32, and is well adapted to good filtration. Practically all stratification took place within the top 2 in., which is the region mainly responsible for the removal of floc and turbidity. At the right is shown graphically the sizes of sand in the filter. The per cent of 48 and 20 mesh was small, and the sand was roughly two-thirds 35 mesh and one-third 28 mesh.

Figure 7 gives the Nyack, N. Y., results. The per cent of 48 mesh is low, but the bulk of this was found in the top 2 in. and practically all in the top 6 in. The stratification of the 35 mesh is evident only in the top 2 inches. A small per cent of 20 and 14 mesh sands were present and these were uniformly distributed from 2 in. below the top to the bottom. This demonstrates in general the advisability of excluding if possible the 14 and perhaps 20 mesh sand entirely. The effective size of the entire layer was 0.50 mm. and the uniformity coefficient 1.38. Since the placement of this combination of sand sizes, the filter has consistently produced an effluent with turbidity quite comparable to distilled water and has not permitted the passage of any carbon applied to the top of the filter. It should be stated that carbon is applied one-half hour after backwashing.

Figure 8 gives Ossining, N. Y. results. Little stratification was evident. The proportion of 20 mesh sand (which was low in the previous filters) was relatively high in this filter. Two inches of fine sand (35 and 48 mesh) were added and resulted in a lowering of turbidity of the effluent from 0.4–0.5 to 0.0–0.2 p.p.m. Over a period of six months, however, much of the 48 mesh sand was lost in backwashing.

Figure 9 for Larchmont, N. Y. shows unusual results. The hydraulic grading was very good, but contrary to the preceding findings, the coarsest sand (28 and 20 mesh) was found at the top, and the finest sand (35 mesh) was predominant at the bottom of the layer. This sand was jet black and coated with 12 per cent of manganese oxide, computed as Mn. The specific gravity was determined for the 35 and 28 mesh sands respectively, and it was found that the larger sizes were lower in weight per unit volume than the smaller sizes due to the build-up of the porous coating. The effect of density was sufficient to cause complete reversal of normal hydraulic sub-

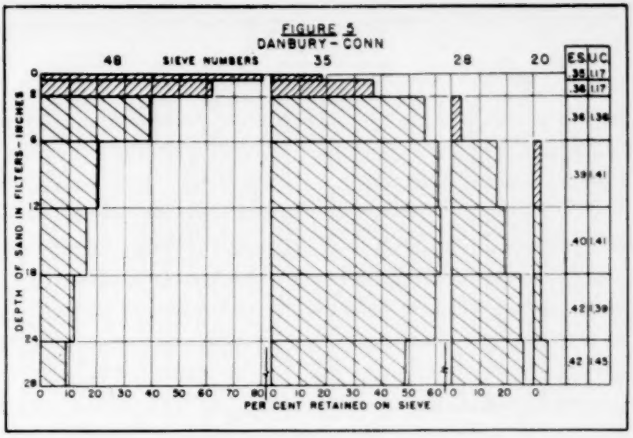


FIGURE 5
DANBURY - CONN
SIEVE NUMBERS
48
35
28
20
E.S.
U.C.
.35 1.17
.36 1.17
.38 1.38
.39 1.41
.40 1.41
.42 1.39
.42 1.45
DEPTH OF SAND IN FILTERS - INCHES
PER CENT RETAINED ON SIEVE

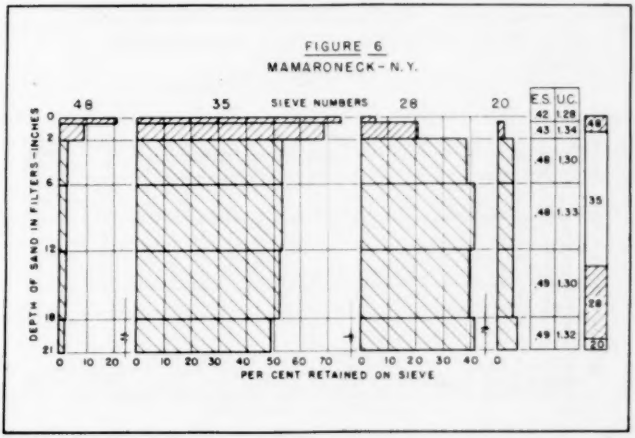


FIGURE 6
MAMARONECK - N.Y.
SIEVE NUMBERS
48
35
28
20
E.S.
U.C.
.42 1.28
.43 1.34
.48 1.30
.48 1.33
.49 1.30
.49 1.32
DEPTH OF SAND IN FILTERS - INCHES
PER CENT RETAINED ON SIEVE

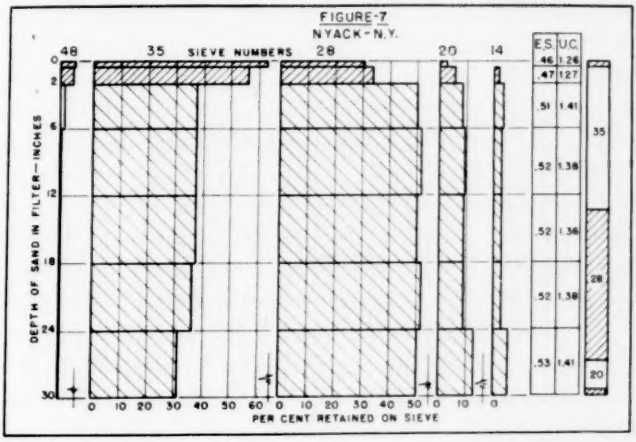


FIGURE 7
NYACK - N.Y.
SIEVE NUMBERS
48
35
28
20
14
E.S.
U.C.
.46 1.26
.47 1.27
.51 1.41
.52 1.38
.52 1.38
.52 1.38
.53 1.41
DEPTH OF SAND IN FILTER - INCHES
PER CENT RETAINED ON SIEVE

FIGURE 8
OSSINING - N.Y.

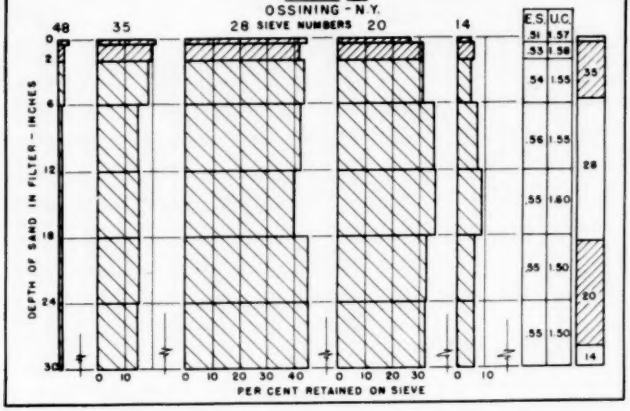


FIGURE 9
LARCHMONT - N.Y.

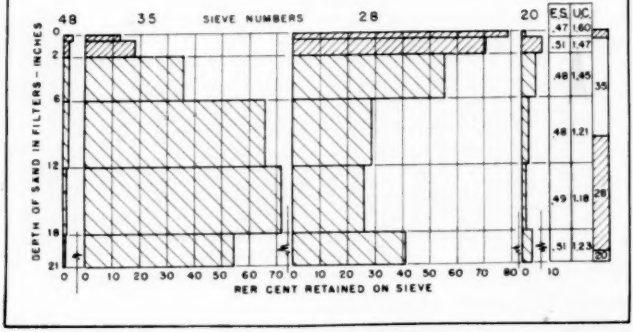
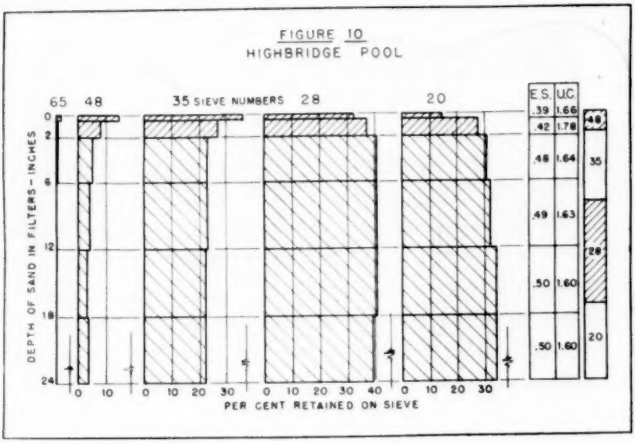


FIGURE 10
HIGHBRIDGE POOL



sidence, but this is in accordance with Stokes' law, which states that the rate of subsidence is proportional to the square of the diameter and to the difference between the specific gravity of sand and water. This filter, however, functions as a contact layer for removal of manganese as well as for filtration.

Figure 10 gives the Highbridge Pool results. This filter was sampled since the lip of the wash water trough is 54 in. above the top of the sand, which is approximately twice the conventional distance. Here we note a relatively large per cent of 20 mesh and a reduction in the 35 and 48 mesh sizes. This naturally greatly increases the uniformity coefficient. No stratification existed below the 2-inch depth.

Comparison of Filter Sands Studied

Figure 11 shows a comparison of all filter sands examined. Effective sizes ranged from 0.38 to 0.54 and uniformity coefficients from 1.32 to 1.67. The Danbury filter, with low effective size and uniformity coefficient, showed the best stratification. The Nyack and Ossining filter underdrains are identical. Both filters have been rebuilt within the past two years. The stratification at Nyack was better than that at Ossining, which may be due to the use of three sizes of sand at the Ossining plant as compared with practically two sizes at Nyack.

Comparison of Commercial Filter Sands

Figure 12 gives typical commercial filter sands produced in New York and New Jersey. Numbers 1 and 2 would give the longest filter runs. Due to the large amount of 20 mesh and the inclusion of some 14 mesh, No. 2 would probably require less backwashing. However, the amount of 35 mesh sand present is too low to provide efficient filtration during times of poor floc formation. The characteristics of these sands are more similar than is indicated by the existing variation between effective sizes and uniformity coefficients.

Number 3 has a larger per cent of 35 mesh, but too great a per cent of 20 mesh. Number 4 is a fair filter sand, although it would be preferable if the 35 mesh were increased, and the 20 mesh decreased.

Numbers 5, 6, 7 and 8 are the best of the lot, and No. 8 would undoubtedly give the clearest effluent, but shortest filter run. It is adapted to a plant with a sedimentation period of 4 to 6 hours. Number 6, with a uniformity coefficient of 1.43 and an effective size

of 0.49 mm., would be best for a 3-hour basin. The small amount of 48 mesh sand would be found largely at the top of the layer.

Numbers 9 and 10 are useful for addition to an existing filter which is lacking in "fines." Number 9 could be added to No. 1 to very good advantage, and would result in a marked improvement of this sand. Number 11 is worthless for filters since it contains too much 100 and 65 mesh. It would increase the uniformity coefficient when added to practically any sand, and at least 22 per cent (representing the 100 and 65 sizes) would soon be lost through backwashing.

Selection of Filter Sand

The bulk of so-called filter sands on the market today are in reality screened for moulding and blasting. The demand for filter sand is exceedingly low and amounts to only two-tenths of 1 per cent of the total sand sold commercially, excluding that used for building, paving and railroad ballast.

In selecting a filter sand it is preferable to take a standard grading rather than have a producer change his screens to meet rigid specifications. The output of several companies should be examined. If necessary, two sands may be blended when the filter is built. If this is done by the producer, the shipment may materially differ from the sample submitted.

In the opinion of the writer, the 48 and 20 mesh sizes should be limited to about 10 per cent each, with the latter excluded if possible. The 35 and 28 mesh sizes should be present in about equal proportions. If a long sedimentation period is provided, the 48 mesh sand may be increased slightly. Sands of this type will have an effective size of about 0.50 mm. and a uniformity coefficient of less than 1.45.

The specifications with regard to sizes suggested by the author are given in Table 2.

The selection of torpedo sand is very important since 35 mesh sand is present next to this fine gravel. If it is too coarse it would seem to provide a ready passage of the 35 mesh material down into the underdrains. On the other hand, it should not be so fine as to be lifted during backwashing. The writer believes that torpedo sand should pass a No. 6, and about 50 per cent each should be retained on No. 8 and No. 10 mesh screens (14 and 20 mesh sand will not remain in place).

In this instance, the maximum diameter of particle is 3.33; minimum, 1.65; and average, 2.5 mm. The average size of 35 mesh sand

is 0.50 mm. The average diameter of torpedo sand is 5 times the diameter of the 35 mesh. From a theoretical standpoint, the maximum ratio to prevent passage of fine sand is 6.4. It would seem desirable therefore to limit the 48 mesh sand to that quantity which will remain in the top 6 in. of the filter.

FIGURE 11

COMPARISON OF FILTER SANDS EXAMINED

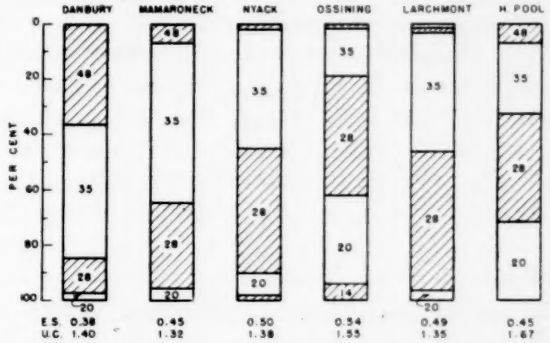
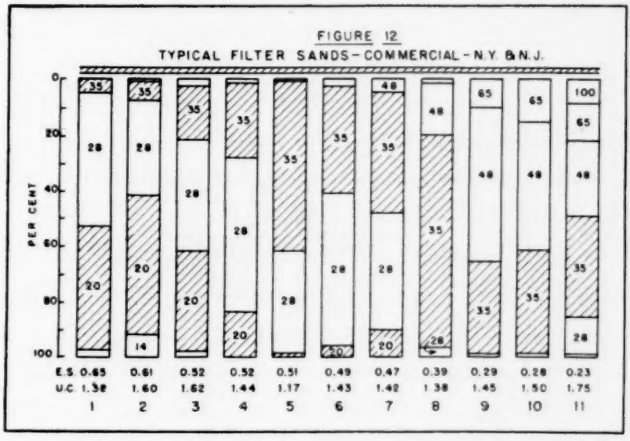


FIGURE 12

TYPICAL FILTER SANDS—COMMERCIAL—N.Y. & N.J.



Removal of Mud Balls

Apparently the formation of mud balls cannot be prevented. Present indications are that a surface wash is capable of breaking down these formations so that they may be removed in backwashing.

For plants not employing this method, however, there is a procedure which is entirely satisfactory. The filter should be washed perfectly clean every two or three weeks. This backwash will require about twice the normal amount of water. When the wash water is clear,

TABLE 2

Suggested Size Specifications

SIEVE NUMBER		SIEVE OPENING	PER CENT RETAINED ON SIEVE	
$\sqrt{2}$ Series Tyler Standard	U. S. Series Equivalent		Minimum	Maximum
		millimeters		
65	70	0.208	0	1
48	50	0.295	0	9
35	40	0.417	40	60
28	30	0.589	40	60
20	20	0.833	0	9
14	16	1.168	0	1

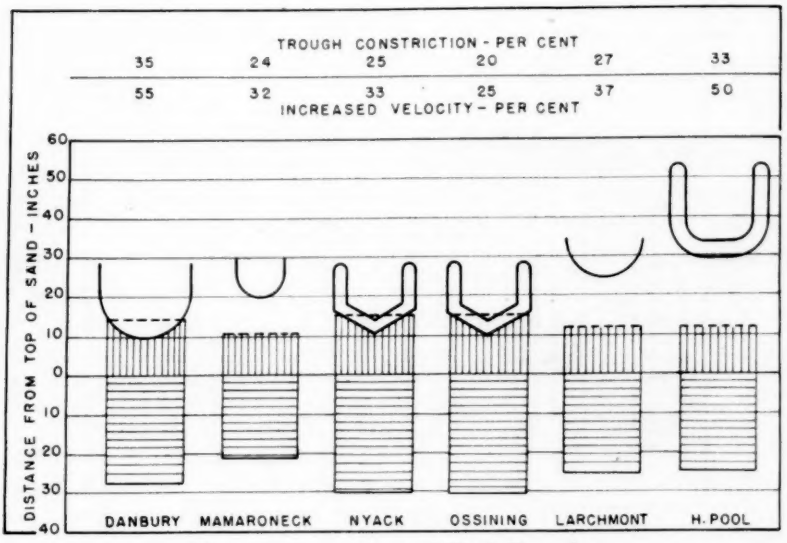


FIG. 13. Comparison of Wash Troughs

the expansion should be dropped to about 20 to 25 per cent and mud balls will be plainly visible on the surface of the sand. They can then be removed manually with a 10 mesh (instead of the conventional 4 mesh) screen attached to a long handle. Mud balls will

penetrate a bed if allowed to remain until they reach a quarter mesh size. The Nyack filter did not show any mud ball formations in the sand samples collected below the surface and a 10 mesh screen has been used for the past year.

Comparison of Wash Water Troughs

Figure 13 gives a comparison of wash water troughs. The Danbury, Nyack and Ossining troughs are placed within the line of 50 per cent expansion. The Mamaroneck and Larchmont filters were rebuilt within the past year using porous underdrain plates. Since these plates were set lower than the previous top gravel line, and the beds were decreased in depth from 30 to 21 and 24 in. respectively, the troughs are now sufficiently high so that a 50 per cent expansion cannot raise the sand to within the trough area. The Highbridge trough is an extreme example of location, the lip being 54 in. above the top of the sand.

The rate of backwash for the Danbury, Nyack and Ossining plants is limited by the loss of sand due to trough constriction. The Mamaroneck and Larchmont filters are limited only by the size of trough and it is impossible to lose sand through backwashing. Needless to say, the maximum expansion at the Highbridge Pool comes far below the bottom of the trough.

Conclusions

1. The finest size of sand that should be used in a rapid sand filter employing 50 per cent expansion is that retained on a 48 mesh sieve.

2. Thirty-five mesh sand (*average* size 0.50 mm.) is required for low turbidity of effluent and to prevent the passage of activated carbon.

3. A 50 per cent expansion of sand is desirable to remove mud balls and maintain the bed in good condition. This requires a rate of backwash of approximately 18 in. at 0°C.; 21 in. at 4°C.; 31.5 in. at 20°C.; and 35.5 in. at 25°C. These rates should be based on gross filter area rather than measured rise at the lip of the wash water trough.

4. The distance from the top of the sand to the *bottom* of the trough should be one-half the depth of the layer, plus a factor of safety of 3 to 6 in.

5. Wash water troughs should be constructed of thin-walled material to give a minimum per cent constriction.

6. In specifying filter sand, it is preferable to use "per cent retained on" rather than "effective size" or "uniformity coefficient."

7. The 48 and 20 mesh sizes should be limited to not more than 10 per cent each, with the latter omitted if possible. The predominant sizes should be 35 and 28 mesh, and these should be present in approximately equal amounts.

8. Torpedo sand or fine gravel should be carefully selected as to size.

9. A 10 mesh long-handled screen should be used for removing mud balls.

Some of the foregoing conclusions may not agree with current procedure but the author believes that they are based upon practical rather than theoretical observations and considerations.

The author wishes to acknowledge with sincere thanks the cooperation and interest of the respective plant superintendents and personnel.

Discussion by *Henry T. Hotchkiss.* * The comprehensive scope of this paper is such that it includes manifold theoretical and practical implications. There is ample evidence to support the hydraulic gradation principle and yet there here appears to be some force acting which tends to distribute the larger sand particles of the fluid mass more or less throughout the entire depth of expanded filtering media. This may be the resultant of an unbalanced kinetic activity in the vertical direction, due to the rise of wash water and sand grain impacts. Certainly, classification is shown to be far from complete after many individual expansions.

Since sand grading was anticipated and since there was no *prima facie* reason for an initial grading or placement in layers over supporting porous plates, the reclaimed sand at the Larchmont plant was simply screened to recover the sand from the gravel and then placed in the filter box. The initial distribution of sand sizes must have been fairly uniform from top to bottom. Such classification as has resulted is entirely due to the effect of washing. The "upside down" condition has been explained as not being inconsistent with Stokes' law on the basis of particle density due to an external coating. The sand, after the removal of this somewhat porous structure, has a normal specific gravity. It would seem that the practical significance of the analysis lies in directing attention to the desirability of chemi-

* Supervising Chemist, Larchmont, N. Y.

cally removing at least a part of the coating on the grains of the upper strata to increase their density and reduce their size. Reasons for maintaining the hydrated manganese oxides as a contact catalytic mass for removing soluble manganese from the raw water have already been advanced. Coated grains tend to become spherical. If the building of a reactive surface does not increase the size beyond particle diameters consistent with desirable filtering characteristics, the performance of the medium as a filter should in no way be impaired. This is verified by experience.

The author indicates that the vertical velocities required to raise torpedo sand are in the neighborhood of 30 ft. per min. (See also Schoklitsch's *Hydraulic Structures*, p. 283, 1937.) It follows that even greater velocities must exist at times when jetting action and other abnormal occurrences take place which displace coarser gravel layers. That such upsets transpire is attested by the number of cases reported. There is no hydraulic regrading under normal conditions because these sizes are unmoved. Conditions at the sand-gravel interface tend to become worse rather than better with every "abnormal" behavior. The critical moments occur just after the wash valve starts to open. Replacement of gravel by porous plates stabilizes the filter at its most vulnerable point. With such a rigid sand-supporting system, sand size and depth become functions of effluent quality and filter run duration. With the evidence before us of the mingling of graded sizes, it may even be desirable, with plates, to select a certain fundamental sand size and approach a uniformity coefficient of one as a practical limit.

By tying together the various functional parts of the filter during the backwashing operation and offering a theoretical discussion of the reason why design should follow and conform to a certain pattern, in order to yield satisfactory practical results, the author has done a real service to the water works field.

Discussion by *Wm. E. Stanley.* * This paper is a timely presentation of interesting data on the distribution of sand sizes in filters after backwashing under service conditions. The general subject of filter sand is of sufficient importance to warrant further consideration.

The author sets up distilled water as the standard of clarity and proposes that this should be obtainable ". . . under the *worst* rather than *average* conditions of floc formation." He concludes, from a

* Professor of Sanitary Engineering, Cornell University, Ithaca, N. Y.

number of experiences of improved clarity with the adding of 35 or 48 mesh sand in certain filters, that sand of this fineness must be used. Possibly these premises should be confirmed. There is some experience tending toward the use of filter sand of a larger effective size. Also there may be some uncertainty as to the value of the endeavor to obtain exceptionally clear water if, perchance, distribution system conditions are such as to result in production of turbidity by chemical precipitation, iron solution or bacterial growth.

The author discusses the relation of sand size and sand expansion to velocity of wash water by comparing filter washing with the rate of settling in still water. The condition of turbulent rise of wash water up through the sand layer does not approximate that of particles falling in quiescent water. The sand grains when acted upon by jets of wash water are lifted to different heights, depending upon their size and shape. When a grain reaches the highest limit to which a jet can lift it, it is shifted to one side into a current of lesser velocity, and settles down to a point where it is again picked up. Thus, there is a continual up-and-down movement in all of which there is conflict with other grains, tending to raise them or to lower them.

The data on the analyses of sand for different depths in a number of operating filter plants are valuable as illustrating the tendency for filter sand to stratify. However, the hydraulic grading of the sand in a filter bed depends upon several factors, the most important being the time in which the wash water valve is closed at the end of the wash. Thus, a sudden closure of the wash water valve will result in a greater mixing together of the sand sizes, than a slow closure. Filters with relatively large amounts of fine sand will be graded by the wash water more perfectly than those containing a well graded mixture of fine and coarse sand.

It seems pertinent to make reference to an extended study of filter sand carried on during the years 1926 to 1935 inclusive by the A.S.C.E. Committee on Filtering Materials for Water and Sewage Works, the summarized results of which were published in A.S.C.E. Proceedings in December, 1936, page 1543.

One series of experiments on glass tube filters included a battery of eight units each with a different top size of sand on which data on filter runs and washing efficiency were obtained from a number of locations over the central and eastern United States. In each case the filter sand was the same, being all sieved and sent out by Mr.

Armstrong at Baltimore while the character of the applied water was different for the several locations.

Another series of tests run at Baltimore and at Toronto included eight glass tube filters each containing sand of a uniform size, the sizes in different filters ranging from 0.37 mm. to 1.75 mm. Determinations of the depth of penetration of floc and silt were made by removing the sand at the end of each run by sections and carefully washing, a quite laborious procedure.

These experimental filters were operated at a filtration rate of 125,000,000 gallons per acre per day and the filter runs were ended when the loss of head reached 8 ft. or the turbidity of the effluent reached 0.2 p.p.m. as measured by Baylis or similar turbidimeter.

A condensation of the summary of the results of these series of experiments is as follows:

1. A good effluent can be secured with any size of sand, provided the filter layer is of proper depth.

2. The depth of a filter layer should be increased as the sand size increases.

3. Hours of service for a properly graded filter, between washings, increase as the size of the sand increases.

4. Efficiency of wash for any given grade of sand is dependent upon the rate of wash and is affected by the viscosity of the water and slightly by the duration of wash. Better efficiencies are obtained with coarse sands.

5. The washing rate should increase with increase of sand size and also with increase in water temperature. A 24-inch rise for fine top sand sizes and 30 in. rise for sands of about 0.6 mm. top size was indicated. Too large a wash rate decreased the washing efficiency.

6. Sand size—top sizes ranging in diameter from 0.6 mm. to 1.00 mm. seemed to give very satisfactory results, while sizes of 0.5 mm. or less gave short runs and were difficult to keep clean.

Discussion by *R. Hulbert.* * Admittedly the question of proper composition and maintenance of the rapid filter sand layer is a controversial one. Mr. Riddick's ideas, expressed concisely in this excellent paper, are plainly the result of experience coupled with careful thought and observation, and deserve every consideration. His discussion of the subject shows a fine blending of theory with practice.

* Assistant Superintendent, Filtration Plant, Detroit.

Here again, one notes that "effective sizes" and "uniformity coefficients" continue to lose prestige as infallible size definitions for rapid filter sand, and instead is found a simple, common sense specification given in terms of percentages retained on certain screens as disclosed by sieve analysis. One perhaps may quarrel (not very much) with the size figures Mr. Riddick recommends, but has no difficulty in understanding just what they mean. Moreover, the finest 10 per cent of the sand selected and the coarsest 40 per cent are not left to chance, as is the case with the usual E.S. and U.C. specification.

Used in conjunction with a layer of torpedo sand on the gravel, it seems that a sand grading comprising the four or five screen sizes suggested, with Nos. 35 and 28 predominating, should nearly always make a satisfactory rapid filter medium. Mr. Riddick's ideal specification, being a happy medium in size between a much finer grading once popular, and the relatively much coarser gradings recommended of late, should generally work out better in practice than either extreme.

The chief problem of filtration is to maintain a perfectly clear effluent all the time. Fine sand on top no doubt helps to prevent the passage of floc during periods of weak coagulation, but sand size by no means always governs the clarity of the effluent. Good flocculation produces filterable water, and without it no sand, of whatever size or depth, can strain out all colloidal and uncoagulated matter at high rapid-filter rates. Another factor, perhaps just as important as sand size, is that of porosity of the filter bed, which is governed largely by sand grain shape, so that different types of sand, which are widely used in filters, would not produce the same porosity even though the size gradings used might be exactly alike.

If there is an optimum size and grading of sand for each particular plant and water, it would seem wise, considering the present state of our knowledge, to try to determine this experimentally rather than set up an arbitrary standard. Filters properly may be designed to take care of the worst conditions, providing that does not place too severe a penalty on design for average conditions; if it does, then it seems best to design for average conditions.

Mr. Riddick has drawn some conclusions regarding the best filter design to improve backwashing results and facilitate maintenance, which are much in accord with those reached as a result of the studies made in Detroit some ten years ago. He regards sand expansion as the best means of regulating the filter wash to insure uniform and efficient results, and again recommends 50 per cent expansion as

about right. Placement of wash water troughs bottoms above the line of 50 per cent expansion is stressed again, which to the writer's mind is most important, since filter sand is never washed out until contact is made with the trough bottoms. The level, V-bottom trough of thin-walled material is certainly preferable, for reasons which he mentions.

Although a symposium on the "when, how and why of mud balls" is long overdue in filtration circles, this is not the time and place to begin one. Are they not formed by and in the rolling mud-scum or "schmutzdecke" during the filter backwash? If so, will not any means of preventing a thick accumulation of this on the filter sand surface, to a large extent, prevent mud ball formation? Anent Mr. Riddick's practical theorizing on interstitial velocities and sand particle equilibriums, why is it that finely divided silt which is lighter than sand, is not washed off the filter surface even at very high rates of backwash? Does the sharp drop in wash velocity as the water emerges from suspended sand surface explain this phenomenon, or is there a better answer?

Enough practical ways of keeping filters free from mud balls have been suggested to permit no excuse for any excessive accumulation of these in any filter layer. Small ones are induced to rise into, and remain in the top sand layers, by shutting the wash valve off very slowly at the end of the backwash, which stops or minimizes jet action and permits all suspended particles to find their natural equilibrium plane, which for small mud balls appears to be in the uppermost sand layers. This makes it possible to screen them out easily at a low wash rate; or as is now done once a year in Detroit plants, they may be broken up by sucking them through an ejector suspended with its mouth at the sand surface, whilst the bed is being backwashed continuously at a very low expansion rate, the muddy discharge from the ejector going back into the wash water rising above the sand.

The subject of rapid sand layer composition and maintenance is always timely and intriguing to filter men. This last contribution of Mr. Riddick's is not only timely and intriguing, but very much worthwhile and very interesting as well.

Discussion by *Edw. S. Hopkins.* * Mr. Riddick's paper is fundamentally a plea for the use of smaller size sand in filter layers.

* Filtration Engineer, Bureau of Water Supply, Baltimore.

The proposition that a sand filter should be expected to remove adequately all suspended material with improper conditioning is somewhat debatable.

The old slow sand filter unit was designed for this purpose; namely, to act as the refining medium with a resultant sacrifice of efficiency in operation. The modern purification plant is expected to condition water in such a manner that a residue of floc is put on the filter layer. This residue is, of course, gelatinous and therefore is entrapped on the surface of the sand grains. This entrapment does not necessarily take place at the surface of the filter, but penetration will occur in direct proportion to the size of sand grain used. This is proven by Armstrong's experiments, in which filters containing sand of uniform size showed maximum penetration of silt into the filter, the maximum penetration being determined when an 8-foot loss of head was reached and the turbidity of the effluent *was not above 0.2 p.p.m.*

Average Sand Size *m.m.*	Maximum Penetration of Silt in Filters *inches*
0.37	3.0
0.43	4.5
0.50	6.0
0.60	9.0
0.75	11.0
0.95	16.0
1.17	24.0
1.75	47.0

This work also proved by visual inspection and by chemical analysis that the compacting of material in the top layers of a *fine* sand filter gave shortened periods of service. When penetration occurred these periods were decidedly increased. If the sand is too coarse, as shown by the table, it is obvious that a normal layer of 24 to 30 in. of sand would pass floc before an 8-foot loss of head was reached.

In view of this most carefully controlled study extending over a period of approximately 10 years, it is not believed that it would be good general practice to decrease the size of sand in filter bed, but rather that emphasis should be placed on conditioning prior to filtration. It will be conceded that if a plant is so poorly designed that it is only possible to produce fragile floc, the filtering efficiency should be sacrificed by decreasing the size of sand. Unless this condition continually exists, an operator is not justified in adopting such procedure.

It has been suggested in this paper that sand filter layers should be

built so as to provide that the top of the 50 per cent expanded layer during washing be at the bottom of the drain trough. Accepting the conventional drain trough as at least 18 in. in overall depth, this would increase the freeboard normally found in a filter by this distance. With a 24-inch filter layer, this would give a freeboard of 30 in. It is questioned whether conglomerations of mud particles would be adequately removed from the filter with wash water velocities equivalent to 50 per cent expansion. It has been generally found that if small mud conglomerations are present on the surface of the layer, decreasing the freeboard will usually remove them. Recent investigations by Turner and others indicate that a maximum freeboard at 50 per cent expansion of 6 in. will provide the best washing conditions. This attitude is taken, recognizing that it is quite true that increased velocity will be encountered in areas adjacent to the wash water troughs. It is believed, however, that the practical effect of a small freeboard offsets any advantage that would be gained by maintaining a theoretically uniform velocity, which would be possible if the top of the expanded bed is kept below the bottom of the wash troughs.

It is unfortunate that the method used for obtaining composite sand samples is rather vague. It is hard to justify the procedure in which different increments were taken for different depths of layer. There appears to be no point in increasing the sample at each increment, since it is apparent that these layers are in direct ratio to each other. This factor makes interpretation of the charts extremely difficult. This is further complicated by the use of sieve numbers instead of actual diameters of sand grain.

It is quite apparent in this study, as is true of many other studies of rapid sand filters, that the effective size and uniformity coefficient do not accurately portray the sand grading in the layer. Since it has been well established that the first few inches of sand layer is the actual filtering medium, it is obvious that the diameter of the top size sand is the controlling factor of the filter. The depth of top size sand that will give efficient removal of suspended material will vary, as previously shown in the table quoted above. Once this depth and size have been established, the composition of the sand beneath it is of minor importance, provided the size grades uniformly. This hypothesis is not in agreement with the principles set forth in the paper, since the author states "the 48 (0.30 mm.) and 20 (0.83 mm.) mesh sieves should be limited to about 10 per cent each, with the latter excluded if possible."

The opinions expressed in the paper would be much stronger if supported by detailed operating data. This is particularly true, since they are at variance with the known results from controlled experiments. It is hoped that these data may be published at some future time, making possible a most careful comparison of the results obtained by such studies with those previously reported, which at the moment adequately support utilization of coarser sands.

Discussion by *Edward R. Stapley*.* Mr. Riddick should be complimented for the concise and lucid presentation of the results of recent tests and observations made by him of rapid filter sands.

In connection with the author's comments on screen ratios and his conclusions thereon it would have been interesting to have seen the sand analysis values listed compared with those obtained by use of a sieve series varying with the fourth root of 2.

Designation of sand sizes by sieve numbers (or meshes per inch) can be but an approximate one. Even with the same nominal wire diameters manufacturers are allowed considerable tolerance in wire diameters and in average and maximum screen openings. Particles which will pass the finer sieves will be found to be larger in proportion to the mesh size than is the case with coarser sieves. Determination of size of sieve separation by measuring a considerable number of last particles to pass, by means of an ocular micrometer or by method of counting and weighing would still seem to be the approved way. Furthermore, it is believed that in all of Hazen's work he used a method of sieve rating in which the mean diameter of the last particles to pass was found. In utilizing his formulae this fact should therefore be borne in mind.

In discussing the hydraulic subsidence of particles as a basis for later prediction of the action of sand particles during filter backwashing, the curve on Fig. 1 representing Stokes' Law has been plotted to include a diameter of sphere of 0.71 mm. Stokes' Law is customarily stated to hold only for particles smaller than 0.085 mm. Below this limit viscosity appears to be the controlling factor in settling velocity, while above it, although viscosity is still active, friction becomes more and more the controlling force. When friction becomes the controlling force, the settling velocity varies as the square root of the particle diameter. For 0.1 to 1.0 mm. particles lying in the transition zone, Allen's Law has been suggested. For sand

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particles of 2.65 specific gravity and a temperature of 10°C., this may be reduced to the approximate simplified formula of $v = 146 (d - 0.00034)$, in centimeter second units, where d = the diameter of the particle and v = its settling velocity. Use of this law would give somewhat higher settling values than those plotted on the author's Fig. I according to Hazen.

The idea of placing wash water troughs with their bottoms a distance above the top of the unexpanded sand bed at least sufficient to clear the greatest sand expansion expected has already been advanced. This idea was mentioned in a paper presented some time ago.[1] The inclusion of a 3- to 6-inch additional height as a safety factor might be worth while.

With the wash water troughs placed at these elevations, the additional constriction presented by troughs constructed with thicker sidewalls, as in the case of concrete rather than steel or other thin material, would seem to have little importance. If anything it would appear that such constriction might result in improved removal of the silt in the wash water due to the somewhat increased velocities between the troughs.

Giving wash water troughs a pitch toward the discharge end would seem to have no real advantage. Such construction, regardless of material used, is bound to be more expensive. The lateral movement of wash water toward the high end, as observed by Mr. Riddick, is undoubtedly the result of lessened friction there combined with a velocity component toward the high end as the rising wash water impinges against the inclined trough bottoms.

Following the author's observations and conclusions on preferred sand sizes and proportions a step farther, the question arises if a sand more nearly uniform even than the sizes and percentages suggested by him might not prove even more efficient as a filtering medium. In selecting a sand, however, it is necessary to consider the economy of obtaining sands of such uniformity, as well as the requirement that the sand layer should not penetrate the supporting gravel layer to any appreciable extent. It is believed that some thought could also be given to the required depth of the sand layer and its relation to the sand size.

Mr. Riddick has given those interested in filter plant design and operation some valuable additional data, and has set forth ideas providing much food for thought.

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[1] Stapley, Edward R., A Discussion of Filter Washing Theory, presented on October 16, 1939, at the Southwest Section Meeting at Dallas, Texas, on October 16, 1939.

Author's Closure: The writer appreciates the discussions, pro and con, which this paper has elicited.

There is considerable divergence of opinion between the views set forth by the Progress Report submitted by Stanley, et al[2] and the writer's findings.

With regard to sand size, the Report states:

"The experiments seem to indicate that sand beds with top sizes ranging in diameter from 0.6 mm. to 1.00 mm. will give very satisfactory results; that sizes of 0.5 mm. and less give short runs, require frequent washing, and are difficult to keep clean. Excellent results can be secured with graded sands having a top size of from 1.00 to 1.75 mm. in diameter, but as the upper limit of size is approached, the depth of bed and rate of wash should be increased beyond those ordinarily used."

The term "top size" seems to have been formulated with the thought that filter sands would be well graded by backwashing. This is not always the case – for instance, the Ossining filter showed rather uniform distribution of 35 mesh sand from top to bottom.

If a coarse sand having a "top size" of 1.5 mm. is used, the rate of backwash (for 50 per cent expansion at 70°F.) would be about 10 ft. per min. With wash water trough constriction now ranging from 25 to 35 per cent for 3-foot rates of backwash, would not troughs for a 10-foot rate occupy almost the entire filter area or be excessively deep? Would not activated carbon tend to pass such a filter if applied directly to the top? And would not the torpedo sand or fine gravel be too greatly disturbed?

Hopkins seems to believe that if adequate facilities are available, any water can be properly treated at all times. Does not weak flocculation occur at some periods of the year regardless of optimum operating conditions? And is this not dependent upon the seasonal changes of raw water rather than upon treatment facilities? At the Ossining plant in particular, where alum is normally employed, there have been times during which no aluminum salt would produce a satisfactory floc, and ferric hydroxide (though forming readily) was extremely fragile.

Experience has shown that filter runs depend as much upon design and sedimentation basin retention as upon the size of sand used.

There may be many plants at which a coarse sand can be utilized to good advantage, but the writer feels that unless this has been definitely established before the construction of filters, sand composed mainly of 35 and 28 mesh material will prove more satisfactory.

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[2] Proc. A.S.C.E., December, 1936.

Effectiveness of Sand Filters in Removing Floc

By G. B. McCall, S. E. Edwards and Edward S. Hopkins

IN 1937, and again in 1938, Baylis[1] presented data indicating that the turbidity test did not necessarily measure the quantity of floc passing through sand filter layers. This conclusion was based upon a study in which effluents from a series of sand filters having layers of varying thicknesses, were passed through cotton plugs at a constant rate and under a constant head for a period of from three to eight days. The plugs were burned and the ash weighed, and it was assumed "that the amount of suspended matter in a particular water is in proportion to the ash." Various coagulants were utilized in this study, among which were alum, iron and lime, Ferrisul, and combinations of these with silica.

A careful inspection of the results of these studies indicated the possibility of some factor causing erratic results, and it was decided to duplicate the work using a different method of procedure.

Five filter units of 1,440 sq.ft. area were chosen for the test. Four of these filters were sand units of varying depths, and the other, an anthrafilt layer. Their characteristics are shown in Table 1. These units were operated as a part of the regular plant procedure for a period beginning with February 9, and continuing until August 19, 1939. The rate of flow was constantly maintained at 2 gal. per sq.ft. per min. The initial and final loss of heads were determined with mercury manometers. Wash water was applied at the rate of 20 gal. per sq.ft. per min. for a period of 4.5 min., until the turbidity reached 75 p.p.m. This procedure was controlled by frequent laboratory testing and was strictly maintained during the experiment. The water temperature varied during the experiment from 37°F. to 70°F.

A paper presented on October 5, 1939, at the Four States Section Meeting at Reading, Pa., by G. B. McCall, Junior Chemist; S. E. Edwards, Principal Sanitary Chemist; and Edward S. Hopkins, Filtration Engineer; Montebello Filters, Baltimore.

[1]Water Works and Sewerage, **84:** 310 (1937); **85:** 855 (1938).

A $\frac{1}{8}$-inch line was tapped into the effluent pipe of the large unit to supply filtered water for the experiment. This line discharged into a 1,000 c.c. Erlenmeyer flask which continuously overflowed, thereby serving as a constant head chamber. Water siphoned from this flask onto a Munktell #0, 130 mm. diameter quantitative filter paper inserted in a Buchner funnel of 135 mm. diameter. The constant head chamber was at a height approximately one foot above the top of the Buchner funnel. Glass orifices were inserted into the discharge line, accurately controlling the flow. Suction was applied to the funnel by a Chapman pump, which maintained the predetermined flow during the test periods. In every instance the test cycle was stopped when the loss of head in the large unit reached 7 or 8 ft., and was not affected by clogging of the filter papers. As shown in Table 2 below, a reasonable number of test cycles were conducted and a large volume of water passed through the paper filters during each operating period. The average rate was 0.27 gal. per sq.ft. per min., which velocity approximated 16 per cent of that passing the

TABLE 1

Filter Layer Characteristics

Filter Number	28	30	32	43*	47
Per Cent Sand 0.17 mm. dia.	0.20	0.10	0.15	0.15	0.00
" " " 0.20 " "	0.00	0.00	0.00	0.00	0.00
" " " 0.26 " "	0.15	0.05	0.00	0.20	0.15
" " " 0.37 " "	0.25	0.10	0.10	0.55	0.50
" " " 0.43 " "	0.50	0.15	0.45	0.60	1.10
" " " 0.50 " "	11.75	1.75	8.55	1.80	6.55
" " " 0.60 " "	0.65	10.15	3.75	2.45	15.10
" " " 0.76 " "	10.80	10.40	13.95	1.50	10.70
" " " 0.95 " "	41.75	39.65	52.70	13.70	27.25
" " " 1.17 " "	10.30	12.50	9.65	11.15	7.10
" " " 1.75 " "	8.00	7.50	5.85	25.00	9.35
" " " 2.50 " "	10.05	7.00	2.35	32.10	15.40
" " " 3.75 " "	4.10	4.10	0.70	10.80	6.80
" " " above 3.75 mm. dia.	1.50	6.55	1.80	0.00	0.00
Topsize sand mm.	0.50	0.60	0.50	0.95	0.50
Effective size mm.	0.48	0.58	0.52	0.80	0.51
Uniformity coefficient	1.89	1.60	1.69	2.27	1.84
Depth sand layer—in inches	22	26	30	28	27
Depth gravel layer—in inches	26	22	18	20	21

* Anthrafilt.

plant units. The amount of material deposited on the paper filters, therefore, represents a reasonable quantity for analysis and is in direct proportion to that passed through the plant units.

The Al_2O_3 content of the precipitated material was gravimetrically determined. Where unusual deposition was noted, a test for iron was also made so as to discount the possibility of error from rust particles being carried onto the filter paper. In making the Al_2O_3 determination, the slightly discolored paper was treated with hot (1:1) dilute hydrochloric acid, and the dissolved aluminum salt subsequently precipitated with ammonium hydroxide at pH 5.5.

It will be noted upon consulting Table 3 that only very small quantities of alum passed through the plant filter units. This averaged 0.0146 p.p.m. as Al_2O_3 or 0.0209 p.p.m. as $Al(OH)_3$. It is fair to assume that this quantity of aluminum hydroxide may be considered equivalent to that of floc. This is much less than the

TABLE 2

Data on Tests

Filter Number	28	30	32	43*	47
Number of Tests	23	22	22	13	18
Gal./Sq.Ft./Min. through Buchner Filter	0.27	0.27	0.28	0.28	0.26
Per Cent Buchner Filter Rate to Plant Unit Rate	14.8	15.3	16.1	17.9	15.5

* Anthrafilt.

concentration detectable by the Baylis or Hellige turbidimeters, which instruments are used to control satisfactorily the appearance of filtered water. The values cited above are somewhat lower than those obtained in the previous studies by Baylis. This, however, in no sense invalidates his ratios or disproves the effectiveness of the use of silicates as shown by his studies.

Contrary to expectation, the concentration of hydrated aluminum oxide did not vary with differences in temperature. This is doubtless due to proper conditioning of the water at all temperatures prior to filtration, thereby once again showing the value of an efficient mixing basin.

The rate of filtration was reasonably constant for all units, averaging 1.70 gal. per sq.ft. per min. The anthrafilt unit gave a length of service approximately three times that of the sand units. Unit

47 gave unusually shortened periods of service, averaging only 49 hours, developing a very high, 3.1 ft., initial loss of head. This was undoubtedly caused by presence of 2.7 per cent of fine sand (0.43 mm. diameter or less) in the top layer, since this actually amounted to 0.8 in. in thickness. The filtered water from the anthrafilt layer had the greatest concentration of floc but was not particularly excessive over that of the sand layers. The thicker sand layer (30 in.) in unit 32 passed the least floc, showing that for experimental work or extreme accuracy of control, this type of sand layer is the most valuable,

TABLE 3

Filter Operating Data

	FILTER 28			FILTER 30			FILTER 32			FILTER 43***			FILTER 47		
	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.
Initial Rate*	2.31	1.69	2.22	2.07	1.78	1.94	2.26	1.69	2.00	2.65	1.59	1.94	2.55	1.64	2.08
Average Rate**	1.94	1.71	1.83	1.90	1.49	1.76	2.15	1.51	1.74	1.69	1.32	1.51	1.80	1.52	1.68
Final Rate*	1.78	0.87	1.13	1.49	1.15	1.31	2.07	1.25	1.48	1.40	0.48	0.93	0.62	0.48	0.55
Initial Loss of Head**	0.80	0.40	0.50	0.90	0.60	0.70	0.90	0.60	0.70	1.07	0.90	0.90	3.10	3.10	3.10
Final Loss of Head**	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	7.00	7.00	7.00	7.00	7.00	7.00
Hours of Service	87.00	30.80	57.30	87.00	34.80	56.80	87.50	41.50	64.60	382.3	114.7	193.0	84.40	23.70	49.10
Liters Per Test Filtered through Buchner Filter	970.9	343.2	645.7	939.6	375.3	642.7	1225.5	473.1	791.7	4128.3	1238.4	2181.6	782.1	255.6	532.8
P.P.M. Al_2O_3 Buchner Filtrate	.0312	.0004	.0165	.0308	.0039	.0165	.0203	.0004	.0091	.0557	.0014	.0179	.0270	.0004	.0130

* Gallons square feet per minute.

** Feet.

*** Anthrafilt.

since the analysis indicated it was 79 per cent more efficient than the more shallow layers in units 28 and 30. However, for practical plant control this efficiency is negligible, since the maximum quantity of hydrated oxide passing the sand units was only 0.0447 p.p.m.

The study shows that properly coagulated water after a sedimentation period of at least 4 hr. will be adequately clarified by sand filter layers. Such floc is sufficiently tenacious even during periods of cold water (37°F.) to be entrapped by sand units and will not pass the layers in any great quantity. The present standard of 0.1

p.p.m. turbidity in the filtered water may be accepted as an indication that the filter layers are functioning properly. For very accurate differentiation, determination of the Al_2O_3 content in the filtered water will reflect the characteristics of the layer used. It is apparent that thick sand layers have an increased efficiency when measured by accurate procedure, but it is questioned if this efficiency is of practical value in relation to plant investment costs and operating procedures.

Since this study is in agreement with the previous one made by Baylis in relation to the rate of floc penetration, these methods may be satisfactorily used as a "yard stick" for the accurate measurement of sand layer efficiencies in respect to depth of layer and size of sand.

The review and criticism by George G. Dobler, Senior Analytical Chemist, Baltimore Bureau of Water Supply, is much appreciated.

Filter Sand—From a Producer's Standpoint

By H. J. Gerrard

IN PLANNING filter installations engineers often wonder why it is so difficult to obtain sand to their specifications. As a producer of industrial sand who is called upon to furnish the various sizes necessary, it is our belief that a brief outline of the method of production and grading of commercial sands might be of some assistance in determining the various standard grades that are available. First of all it is necessary to realize that sand is not a manufactured article, but is a product of nature that is taken from the ground in its crude state, with a conglomeration of all sizes. With this in mind, the producer must devise a method of preparing the sand for industrial use. One of the largest markets for silica sand (in the grain sizes comparable to filter sand) is the foundry industry. Therefore, grading must be regulated so as to take care of the various sizes that are applicable to foundry requirements. The procedure of production of that type of sand at one of our plants is as follows:

The crude sand, ranging in sizes finer than 270 mesh to pebbles of $\frac{3}{4}$-inch diameter or larger, is pumped through a pipe line from an inland lake or pond to the plant. The first operation carries this crude sand through a scrubbing machine where all the dirt, clay and vegetable matter are removed. The sand then passes through a series of scalping machines where the $\frac{3}{4}$-inch and larger, then $\frac{5}{8}$-inch, and then the $\frac{1}{4}$-inch sizes are removed. The balance of the sand then passes through classifiers where the fine sand is separated from the coarser grades, and passes to a purifying plant to be prepared for the glass industry. The coarser sand is then taken by belt to a storage pile where it drains for 2 or 3 days.

A paper contributed by H. J. Gerrard of Whitehead Brothers Co., New York.

EDITOR'S NOTE: The standard procedures to which reference is made in this paper are found in the publication "Testing and Grading Foundry Sands and Clays, Standard and Tentative Standards" of the American Foundrymen's Association. The fourth edition of this manual was published in 1938.

This coarser sand is then elevated to a drying tower where it passes through a series of steam pipes by gravity feed. This evaporates all of the moisture and leaves the sand bone dry. The sand is then fed directly to the screening tower where it passes over a series of inclined mechanical vibrating screens and the various sizes necessary for the foundry trade are separated.

Each size as it comes off the screen is fed directly to the storage space reserved for that particular grade. Bin spaces are regulated to accommodate that particular percentage of each size produced through a normal run. Five separations are made in order to produce a balanced output for the plant, graded to the sizes that are commercially known as #00, #0, #1, #2, and #3. The #00 or the finest grain size is generally used for foundry core work or steel molding sand; the coarser grades for blast cleaning. An average screen test, according to the American Foundrymen's Association procedure will show the following:

PER CENT RETAINED ON U. S. STANDARD SIEVES	GRADES				
	#00	#0	#1	#2	#3
6	—	—	—	—	6.4
12	—	—	—	20.4	86.4
20	—	—	17.8	79.4	7.2
30	—	2.0	55.8	0.2	—
40	0.2	30.2	23.8	—	—
50	2.6	37.6	2.4	—	—
70	26.4	22.4	0.2	—	—
100	47.8	6.6	—	—	—
140	17.0	1.0	—	—	—
200	5.2	0.2	—	—	—
270	0.6	—	—	—	—
Pan	0.2	—	—	—	—
*Effective Size	0.12	0.22	0.51	1.32	—
*Uniformity Coefficient	1.66	1.82	1.49	1.22	—

* Not figured or shown on a regular foundry sand analysis; merely added to show the grading according to the usual filter specifications.

The above sizes are subject to a tolerance of plus or minus 10 per cent of each percentage retained on each individual screen, and have been accepted as a standard by the foundry industry. The sizes of blast sand specified by the foundries are those which give the

greatest efficiency in conjunction with their sand blasting system. One will note from the above table that only the "percentages retained" on the various screens are shown. This we believe gives a true picture of the exact grain distribution.

The production of filter sand, while a desirable business, is of such small volume that it is impractical for commercial sand plants to consider special screenings to obtain specific sizes other than standard, since it would automatically make special grades of the balance of the sands. This would necessitate additional storage space for the special grades thus produced. It would also make it necessary to create a new market for these special grades. One can readily see how costs would mount and how it would interrupt the production of the material for which there is a ready market.

As a producer, it is the writer's opinion that inasmuch as all foundry sands are graded in the same manner, it would be to the advantage of water works engineers to study the American Foundrymen's Association method of sand analysis with view to considering its adaptation to their specifications. From the clear picture shown by these analyses, where the actual amount of each size retained on each screen is given, the engineer can easily determine whether or not a sand is satisfactory. In addition to this, were he to desire something slightly different, he could select two gradings which when blended would produce the required size.

The terms "effective size" and "uniformity coefficient" do not give the producer a clear picture of the sieve sizes required, since both of these values must be determined by the engineer from a plot of "per cent passing" the sieves rather than by the "per cent retained" which is the basis of the A. F. A. rating. The lack of understanding between producer and engineer may make it necessary to purchase a sand produced at a distant point with high delivery cost, whereas a blend of local sands might give as satisfactory results at a considerable saving.

ABSTRACTS OF WATER WORKS LITERATURE

Key. 31: 481 (Mar. '39) indicates volume 31, page 481, issue dated March 1939. If the publication is paged by issues, **31:** 3: 481 (Mar. '39) indicates volume 31, number 3, page 481. Material enclosed in starred brackets, ***[ ]***, is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.—Bulletin of Hygiene (British); C. A.—Chemical Abstracts; P. H. E. A.—Public Health Engineering Abstracts; W. P. R.—Water Pollution Research (British); I. M.—Institute of Metals (British).*

PUMPING

A History of Rotary Engines and Pumps. ANON. The Engr. (Br.). *In 21 installments beginning* **167:** 43 (Jan. 13, '39), and closing **167:** 669 (Jun. 2, '39). The history is confined to events preceding the 20th century, which latter are expected to be recorded in subsequent contribution. Article is detailed and exhaustive. One of earliest known rotary pumps is ascribed to Grollier de Serviere or to Pappenheim, described as early as 1636. Another celebrated pump described by Remelli in "Le Diverse et Arteficiose Machine" in 1588. The third of the ancient pumps, known as an "abutment pump" is so old that original inventor is unknown. Earliest rotary engine about which sufficient is known to merit attention is described by Amontons in 1699 as a "moulin-en-feu." In 1723 Leipold so modified Amonton's engine that later device bears his name. It was toward end of 18th century that Watt was the first to think of the rotary engine which he mentions in a letter to Roebuck in Feb. 1766. This engine was patented in 1769. By 1780 Boulton and Small still felt that it was superior to reciprocating machine, but Watt was rapidly losing faith in it and in 1782 patented another form of rotary engine which seems also to have been conceived by Cameron who, in turn, patented a rotary engine in 1784. Among the more notable of the engineers of Watt's day who were taken with the "rotary fever" was Jonathan Hornblower who patented his rotary engine in 1781. Bramah, one of greatest mechanics England ever produced, patented his first rotary engine in 1785. It greatly resembled Watt's engine of 1782. Working model of a wheel engine was exhibited before the Irish Academy in 1789 by Cooke. About this time (1791) Ridge proposed rotary pump typified by six gear teeth on each wheel, and another type almost precisely similar to that "invented" 75 years later by Root. (All these schemes were reinvented again and again.) Rumsey (1792) proposed an engine

consisting of "two wheels of metal exactly circular or elliptical with hollows and rounds, i.e. cogs" which were to intermesh and be driven around by steam. In 1799 Murdock proposed rotary engine similar to those already invented by others. It is noted that these schemes were frequently reinvented, an example being Murdock's engine exhibited before the Inst. of Mechanical Engineers as late as 1850. Flood of inventions of rotary engines in the late 18th and early 19th centuries is partly justified by the lack of faith in the crank mechanism which is epitomized by work of Stephenson on the motion of crank and connecting rod. Even after 50 years of use of the crank in engines, some believed it was responsible for waste of much power from steam. By 1850 weight of the concerted opinions of Russell, Galloway, Stephenson, and others began to convince engineers of futility of arguments advanced against the crank. Inventors of rotaries found themselves forced to find other justification for rotary principle. Today one of chief advantages of the rotary is its ability to run at high speeds. Many early rotary engines and pumps failed for purely mechanical reasons, although theoretically they are perfect machines. Today difficulties of packing and uneven wear are avoided rather than overcome. For certain pumping duties the unique combination provided by a rotary of the positive-acting characteristics of the reciprocator, with the capacity of the centrifugal for running at high speed and giving continuous output, is of great value. During the 19th century invention and design of rotaries is jumbled and chaotic, as typified by the list of machines bearing such names as "rotary-piston," "eccentric," "crescent," "roots," "spherical," "concealed reciprocating," etc. One of simplest forms of the rotary piston design is that in which a drum turns concentrically in a cylinder of large diameter and carries a "piston" reaching across the annular space between the drum and cylinder. Still quite commonly, although incorrectly, assumed that rotary piston engine has no dead points similar to those found in the reciprocating engine. In fact rotaries are nowadays more important as pumps than as engines. Now possible to design them with either constant or variable delivery. About twenty of the rotary piston machines invented in the 19th century are described in detail. All eccentric rotaries used as pumps have similar characteristics; slight differences arising only from the differing actions of abutments hinged to the cylinder or sliding in the cylinder. Selected number of such pumps described in detail. Shipton's engine, which is clearly a rotary, was unsuccessful. Shipton also made two designs with oscillating cylinders. Both were failures. Crescent rotaries, like the eccentric machines, constitute a special case of the rotary piston. In them, piston is situated in the cylinder and consists of a crescent covering the whole of the cylinder walls. Machines of this type as pumps or compressors can deliver a volume per revolution substantially greater than the swept volume of the cylinder. As an engine, machine is notable since it can provide for expansion of the steam without external valves. Majority of patents for Crescent machines relate to devices in which blades slide in and out of the rotor. Some designs are described, with particular detail of engine of J. T. Beals. Stephenson attempted to use one of these engines on a boat with disastrous results. Wimshurst was among first to connect engine to a screw for driving sea-going vessel. Early "futilitarians," and even moderns who are not inhibited by

knowledge of thermodynamics and mechanics involved continue to dream of "wheel" machines. Earliest of wheel engines was invented before 1800. Cooke's is outstanding, driven around purely by the pressure of the steam, somewhat on the order of the Pelton wheel. Some of the many engines described have a distinct resemblance to a well-known form of perpetual motion machine. In 1879 L. B. Lawrence patented a wheel machine in the United States. "Differential" machine is distinguished by absence of an abutment. In these machines, pistons move with varying velocity as distinct from complete rotary motion or stop-and-go motion of a true reciprocating machine. They have all the disadvantages, however, of long reciprocating machines. Only three types of differential machines are known. Machine suggested by Mead in 1808 being among the earliest. From the middle of the 19th century it became better recognized that machines in which movement of pistons never actually ceased were more practical than the stop-and-go variety, and various modifications of Mead's and Smith's engines were suggested. Among these may be included the machines of Thomas, 1812; George Smith, 1870; Stewart, 1883; and the Rota engine of 1889. It is not necessary to be restricted to a single rotor in a rotary engine or pump. Two or more may be used and a large number of bi-rotor machines were proposed during the 19th century. The two rotors act mutually towards each other as abutments and there is, therefore, no need to provide separately for some moving part to act in that capacity. Best-known modern bi-rotors are the simple gear pump and the Root blower, with the advantage of compactness for any given output. In studying characteristics of such a pump it will be noted that form of delivery curve depends on the shape of teeth and the path of point of contact. Difference between max. and min. delivery varies as square of the tooth height; the larger the number of teeth the smaller their height and, hence, the smoother the delivery. Intricacy of design of such pumps results from conflict of heat and hydraulic requirements. An infinite number of curves are available which will permit the mating of gear teeth, the most common being the involute, but this shape requires considerable undercutting of the tooth. The suggestion of use of helical gears has been made to overcome some difficulties of straight tooth pinions. Trapping would thus be eliminated since path of escape would always lie open along the bottom of the tooth space and substantially constant delivery could be obtained by making the face width equal to multiple of axial pitch, since with such an arrangement all the phases of the engagement of the two teeth are present at the same time. Excepting only in particulars of shape of teeth there is little of special interest to distinguish one gear machine from another. Six gear-tooth machines are described in detail, beginning with the Wilson machine of 1802. From practical point of view Root machines have the advantage that they are simple to manufacture. Since they can be made so that rotors neither come into actual contact with one another nor with the casing, they are reliable in use and gifted with more life than is usual among rotaries. The Root dumb-bell type rotor gives cyclic variation of delivery occurring four times per revolution, resulting in a characteristic beat. Twenty years before Root machine was shown in Paris in 1867, similar machines were manufactured in Birmingham. By 1877 it was found that machines with H-rotors could be more easily manufactured and were

less easily damaged. After 1877 little change took place in the design of Root blowers. Not all bi-rotor mechanisms can be classified under the heading "gear" or "Root." A large number are included under classification of "intermeshing" because, whatever the detailed nature of design, projections on one rotor intermesh with slots in the other. Quite a large number of such machines attained some success during the 19th century, of which nine are described in detail. In a large class of machines, named "concealed reciprocators" conventional reciprocating mechanisms are used but the various members are rearranged and caused to have unusual motions. Earliest of the machines of this type patented by Witty in 1811. A most interesting group of machines is termed "spherical" because working chamber has that shape. Many machines of this group have proven practicable and successful. All are, in fact, reciprocating—which accounts, in part, for their relative success. Their chief advantage is compactness for given power output. Their weakness is difficulty in packing. The first of this type to be patented is the Dakeyne (1830) machine. Up to and including 1850, five other machines of this type had been patented. First of a group of engines involving the use of universal joint was patented by Beauchamp Tower in 1885, at time when advent of the dynamo had encouraged the development of a small high speed prime mover. At American Institute Fair, held in New York in 1874, series of trials of rotary engines was conducted on four machines showing variations in water rates from 93 to 279 lb. of steam per indicated h. p. hr. By end of the 19th century very few rotaries had proven successful. In the 20th century others have been patented, some of which have been invented and patented several times before.

Part II. *Ibid. In 9 installments beginning* **168:** 90 (Jul. 28, '39), *and closing* **168:** 294 (Sep. 22, '39). The hopelessness of the invention of a satisfactory rotary steam engine is realized by all except the few futilitarians who are still striving in this direction. Inventors are also hopelessly at work attempting to devise a rotary, internal combustion engine. Better-informed inventors are devoting attention to rotary pumps, blowers, compressors, meters, and compressed air motors. Rotaries at present find their principal applications for lower power loads where their special qualities outweigh their relatively low efficiencies. Applications come principally in air compression; movement of hot, viscous liquids against high pressures, as in oil fields; and for machines with absolutely uniform discharge. Among eccentric machines, few novel suggestions have been made in present century, the last of the type having gone out of production within the last few months. Practically all modern rotary compressors for pressure above 5 lb. per sq. in. appear to be of the Crescent type. Modern gear pumps are made in capacities up to about 1,000 g.p.m. Internal gear machines are made in capacities that are very large relative to those usual among small rotaries. Pumps with capacities up to 46,200 gal. per hr. (Imp.) running at 250 r.p.m. are made. Mono pump appears to be a form of screw pump and screw pumps in large sizes are now able to compete successfully in certain services with centrifugal and piston machines. There has been very little change in Root machines since the end of the last century and they are now used exclusively for blowers. Concealed reciprocator machines have achieved a considerable degree of importance as hydraulic pumps and motors, particularly when linked together to form an hydraulic transmission gear unit.—*H. E. Babbitt.*

HYDRAULICS

Pitot Tubes in Large Pipes. EDW. S. COLE AND E. SHAW COLE. Trans. A. S. M. E. **61**: 6: 465 (Aug. '39). Previous tests (Trans. A. S. M. E. **57**: 281 (Aug. '35)) on 12″ line proved that effect of angular flows past Pitot tubes was negligible. Present work was on 40″ and 78″ steel penstock. Forty in. pipe tests were checked with weigh tanks. Tests were run with the Cole reversible Pitometer and with a heavy duty type Pitometer. The value of velocity traverses and pipe factor, that is, the ratio of mean to center velocity, as a means of determining the flow pattern in a conduit has not been fully appreciated, and any theory regarding turbulent flow must take into account the behavior of this pattern. For good gaging points, traverse curves are smooth and their pipe factors constant over a wide range of velocities. Smooth traverses do not have "humps" or irregularities, and in a rough pipe they take the general form of a parabola. Irregular curves have been found in pipes of all diameters and have been an indication of disturbances a short distance upstream. Upstream contractions give flat velocity curves which are smooth if the contraction is gradual as in a Venturi throat. They are irregular if abrupt changes in section are present above the gaging section. Accurate tests can be made when the velocity distribution is carefully determined, for at a good gaging point the pattern will have a form which remains constant even though unsymmetrical. Although the pipe factor under such conditions remains constant and can be reproduced on different days, as was done for most of the traverses in the large pipes, it is of course closely related to pipe roughness and for this reason should be checked if too long a time elapses between tests. The change in roughness is usually a slow process, but in one unusual case where a pipe had recently been cleaned an appreciable change in pipe factor was observed in less than two months. Where stable flow exists at the gaging section, Pitometer measurements of discharge can be made with a high degree of accuracy in large pipes. It is possible to use either the conventional velocity-traverse method at each discharge or to determine the pipe factor and make subsequent measurements of the center velocity only.—*Homer Rupard.*

Investigation of Errors of Pitot Tubes. C. W. HUBBARD. Trans. A. S. M. E. **61**: 6: 477 (Aug. '39). In late years some hydraulic engineers have come to distrust the Pitot tube as an accurate instrument for measuring the flow of water in pipes. It has been found that some Pitot tubes do not have the same coefficient for all conditions of flow. There is, therefore, general disagreement as to the coefficient which must be applied for a particular set of conditions to obtain the velocity within 1% or less of the true value. Experiments were made in an effort to find the reasons for this variation of the coefficient. Tests made on an 84′ rotating boom in still water showed the coefficient of the impact tip to be unity. The Pitot deflection was balanced against a centrifugal head. A method of measuring the angularity of turbulent flow in pipes by use of the law of probability was devised. The measurements made in pipes 12″, 40″, and 78″ in diameter check the Prandtl and von Karman theories of turbulent flow closely and show the mean angularity to be about 2 deg. Tests made to determine the effect of the Pitot support rod showed that the support rod did

not cause an error in the impact-tip reading, but did effect the pressure measured by the wall piezometer. The rod was also found to effect the reading of the pressure orifice of the Pitot-static tube. It follows that any form of combined type of Pitot tube in which the error could be determined by calibration of the instrument alone would give accurate results. The average magnitude of pulsations was found to be too small to cause an error in reading of the Pitot tube. Conclusions drawn from these tests were that the impact tip correctly registered the true dynamic head whether the flow was smooth or turbulent, but that the pressure piezometers were likely to be in error. Since the pressure piezometer of the Pitot-static tube was a part of the instrument and calibrated with it, this instrument was considered more reliable than the simple Pitot tube using wall piezometers.—*Homer Rupard.*

Water Hammer. HENRY RYON. W. W. & Sew. **86:** 169 (May '39). Difficulties with lead joint failures in 4½ mi. force main at Dannemora, N. Y., led to this study. Normal operating pressure is 275 lbs. per sq. in. When pump stops, hammer causes max. of 370 lbs., min. 90 lbs. Simple description of water hammer waves is given. Observed head checks closely with Joukovsky's formula. Pressure relief valve and air chamber proved inadequate. Cone valve was installed on discharge and is operated against running pump. Slow opening and closing action of this valve provides effective hammer elimination except in case of power failure.—*H. E. Hudson, Jr.*

Reduction of Shock Pressure in Solvent Delivery Lines. HOWARD S. GARDNER AND JOHN H. FOLWELL. Ind. Eng. Chem. **31:** 446 (Apr. '39). The simplest theory of shock pressure (water hammer) is based on the instantaneous closure of a valve against a column of liquid which has been flowing through a pipe under constant conditions. The pressure can be calculated from the expression

$$P = V\sqrt{\frac{\rho k EL}{g(EL + Dk)}} = \frac{V}{\sqrt{\frac{g}{\rho}\left(\frac{1}{k} + \frac{D}{EL}\right)}} = \frac{a\rho}{g}V$$

in which a is velocity of shock pressure wave, ft./sec.; g is gravity, ft./sec.^{2}; k is bulk modulus of elasticity of pipe material, lb./sq. ft.; E is modulus of elasticity of pipe material, lb./sq. ft.; L is thickness of pipe wall, ft.; ρ is density of liquid, lb./cu. ft.; D is internal diam. of pipe, ft.; V is extinguished velocity, ft./sec.; and P is the shock pressure in excess of normal pressure, lb./sq. ft. The max. shock pressure will be developed in the line when the time of valve closure is less than the time for a round trip of the pressure wave in the line. This critical time is $t_c = 2N/a$, in which t_c is time for round trip of pressure wave, sec., and N is length of pipe from valve to intake, ft. In tests on a pipe line consisting of 3,000′ of 3″ pipe with 250′ of 2″ line on the delivery end, conveying lacquer solvents, shock pressures up to 200 lb./sq. in. were observed, resulting in leakage and bursting of the line. The shock pressure and the normal pressure, in themselves, were insufficient to overstress the wall of the pipe. Assumed that failure was due to the almost instantaneous impact of the applied shock pressure, and possibly to mild corrosion fatigue.—*H. E. Babbitt.*

Dimensional Analysis and Performance of Centrifugal Pumps and Fans. J. JENNINGS. The Engr. (Br.) **167:** 614 (May 19, '39). The method of dimensional analysis is useful when applied to analysis of performance of centrifugal machines. The method consists of sorting out groups of variable quantities such that the physical dimension of these groups mutually cancel, and then expressing them as functions of one another. The number of groups that are independent of each other in any given equation is limited to the difference between the number of variables and the number of independent fundamental dimensions used. Among advantages of the method may be included: characteristic curves are directly comparable, one with another; effect of viscosity or roughness could be investigated independently of other variable quantities; pump and fan similarity laws are embodied in the curves and need not be considered separately; the relation between experiments on models and on the operation of full-scale machines can be examined easily; and the operation of pumps with fluids of varying density and viscosity may be more easily forecast.—*H. E. Babbitt.*

Characteristics of Pipe Lines and Pumps. HELLMUT GOTTING. Gas-u. Wasser. **82:** 120 (Feb. 18, '39). Characteristic lines for the function $h = f(\vartheta)$ are developed for pipe lines, using the friction law $h = k \frac{V^{1.77}}{D^{1.23}}$, and for pumps using results of test. The characteristic lines for pipe lines are parabolically rising curves starting, for $\vartheta = 0$, tangentially to the static head of the system. The curves for the pumps are parabolic with a maximum point, and only the falling branch can be used. The curves for the combined action of several pumps are constructed by adding the abscissas of the curves for the individual pumps. The point of intersection of a set of the two types of curves shows the condition $h = f(\vartheta)$ existing in the pump-pipe system. By constructing the curves for different pumps and pump combinations and also for different pipe lines the conditions existing for the various requirements of the distribution systems can be predicted. Efficiency curves of the pumps can be combined with this study to find the most economical selection of pumps and pipe diameters.—*Max Suter.*

Jet Deflectors for High-dam Outlet Conduits. HAROLD A. THOMAS AND WALLIS S. HAMILTON. Civ. Eng. **9:** 297 (May '39). Methods of minimizing the erosive potentialities of large jets have been studied at the Carnegie Inst. of Tech. by means of models, with conclusions as to designs which may aid in the partial dissipation of as much as 500,000 hp. of energy, as at the Tygart Dam.—*H. E. Babbitt.*

Clarification of Water. J. A. BUCH. Rev. soc. cubana ing. **31:** 428 ('38). Tests on scale models (construction not shown) of settling tanks in aqueducts indicated that velocity and pressure heads may be interconverted with negligible loss, provided that at no point the acceleration of the water exceeds about 0.194 m./sec.^{2} In settling tanks it is important to eliminate eddy currents, such as are produced by too rapid expansion in stream cross section. —*C. A.*

Leonardo da Vinci's Studies in Hydraulics. FILIPPO ARREDI. Ann. Lavori Pub. (Rome) **77**: 357 (Apr. '39). In Leonardo's time the education of the artist ranged all the way from the fine arts proper to engineering construction and hydraulics. Evidence of his keen interest in the last-mentioned abounds throughout the writings of practically his whole life; but he made no attempt to assemble these studies. He can be regarded as the discoverer of some fundamental laws, as the student of problems the mere enunciation of which marked a great step in advance, as an observer of singular penetration, as the founder of a new technique based upon laws and observations, and as a skilful engineer. His manuscripts after his death were not easy of access nor easy to decipher, so it has happened that to others who, many years after, rediscovered some of his laws independently, has gone the credit of priority. Author gives 32 quotations from the manuscripts and 10 facsimile reproductions of portions of them. Further articles regarding Leonardo da Vinci's studies, and inauguration of the Leonardi Aula in the Ambrosiana at Milan on Mar. 31, '38 are given *ibid.* **77**: 462, 532 (May '39).—*Frank Hannan.*

Ancient Rome and Modern Technology. On the Screw of Archimedes as Described by Vitruvius. LUIGI JACONO. Excerpt from "Notizzie degli Scavi" a cura della R. Academia dei Lincei. Reproduced in Ann. Lavori Pub. (Rome) **77**: 529 (May '39). The description is readily followed. Galileo remarks in his Mecchaniche that the invention of Archimedes is not only wonderful, but even miraculous; for we find the water ascending the screw although it is falling all the time. Although adopted universally, especially in Egypt for raising the Nile water, no account could be found of how the power was applied. Many suggestions, more or less ingenious, some of them illustrated, have been offered. The mystery has now been solved by a fresco (reproduction given) unearthed in Pompei picturing a slave at work on the outer cylinder of the screw as on a treadmill, with the water gushing out.—*Frank Hannan.*

Fluid Flow Research. S. R. BEITLER. Ohio State Univ., Eng. Exp. Sta. News **11**: 3: 9 (Jun. '39). Accuracy of measurements being considered in all countries; one result has been increase in accuracy of measurements and showing of limitations in present knowledge. Standard devices used to measure large volumes of fluid of 3 types: sharp-edged orifices, rounded approach orifices commonly called flow nozzles and Venturi tubes. Study by committee of second type now in progress, general preliminary conclusions from co-operative test results are that tolerances of measurement will have to be larger for nozzles than for sharp-edged orifices; will be possible to make slightly more accurate measurements with sharp-edged orifice than with flow nozzle. Venturi tubes, first primary elements, have had less known about them than other elements and should be studied.—*Martin E. Flentje.*

Submerged Weir Formula Verified. GLEN N. COX. Eng. News-Rec. **123**: 272 (Aug. 31, '39). General formula developed at conclusion of some 2700 tests at Univ. of Wisconsin on flow over submerged weirs of sharp-crested and ogee type. Water flowing over submerged weir forms trough downstream from crest and this is followed by some rise in water surface. Found advanta-

geous to measure downstream head in trough. Nappe either plunges below surface at weir or remains on surface. First type of flow is more stable for low submergences and second for high: there is considerable overlap in which either type may occur. Discharge for given head and submergence is quite different for the 2 types of flow: 2 general equations are necessary therefore. General formula, $Q = CLH^n$ was used. Both C and n varied for ogee weirs. Derived equation for ogee vertical back weir is as follows:

$$\text{For nappe under, } C = 3.895 - \frac{0.433\,S^3}{1.35 - S}$$

$$\text{and} \qquad n = 1.625 - \frac{0.140\,S^2}{3.0 - S}$$

$$\text{For nappe over, } C = 3.895 + 0.01064\,P - \frac{(1.52 - 0.1886\sqrt{P})\,S^3}{1.433 - 0.01353\,P - S}$$

$$\text{and} \qquad n = 1.625 + \frac{0.036}{\sqrt{P}} - \frac{\left(\dfrac{0.428}{P - 0.169} - 0.0129\right)S^2}{0.886 + \dfrac{1.31}{P} - S}$$

Where C = coeff. in $Q = CLH^n$, L = length of crest in ft., H = observed upstream head plus $V^2/2g$, n = an exponent, P = height of weir crest in ft., S = submergence expressed as a decimal. Hydraulic data on low concrete dam of Miami Conservancy District (Eng. News-Rec. **120:** 137 (Jan. 27, '38)) furnished opportunity for testing formula on higher weir and much greater heads than those used in tests. Max. difference between observed and calculated upstream heads is 0.16′ and occurs for upstream head of 2.4′. This difference is less than 7%. Numerical difference decreased for further increase in head and amounted to 0.10′ with head of 5.85′, i.e., 1.7%.—*R. E. Thompson.*

Analysis of Water Hammer in a Typical Colorado River Aqueduct Pumping Plant. R. M. PEABODY. Trans. A. S. M. E. **61:** 2: 117 (Feb. '39). The behavior of a pumping system during a transient is the result of interaction between the pump, the pipe line, the check or shutoff valve, and, in many cases, also a relief valve, air chamber, or surge suppressor. If the characteristics of each of these elements are known, the conditions during the transient can be quite closely predicted. The formulas for the pipe line, even including branch pipes, changes of section and dead ends are well supported by experimental verification, as brought out in the previous symposium on water hammer. Little has been published on the reverse-flow characteristics of pumps. It appears quite probable that characteristic diagrams for pumps of a given specific speed will be found similar. However, information on complete pump characteristics is generally available for only a limited range of specific speeds. Reliable data on the loss of head in valves as a function of the valve opening are almost completely lacking. Analysis by the methods outlined involves a great deal of laborious calculation, but approximate simple formulas that are generally applicable are totally lacking. Much time and labor can be saved

and a clearer picture shown of the pressure and discharge by using the graphic method of water-hammer analysis for the pipe line head variation. The pump diagram, the valve characteristic, and the inertia equation must be coordinated with the water-hammer diagram. The necessity for step-by-step and trial-and-error calculations is not entirely eliminated, but the work is greatly simplified. For important installations, the various transients possible of occurrence must all be analyzed and the system constructed to be safe for the most severe conditions reasonably probable. The cases used as illustrations represent approximately the condition at one of the pumping plants on the Colorado R. Aqueduct. For the purposes of this paper, the effects of compound and branching delivery pipes, the inlet pipes, and other conditions peculiar to those plants have been omitted, and the examples do not accurately represent actual conditions at any of the plants.—*Homer Rupard.*

Relation of the Statistical Theory of Turbulence to Hydraulics. A. A. KALINSKE. Proc. A. S. C. E. **65:** 1387 (Oct. '39). A fluid at any point is in a state of turbulence if the direction and magnitude of its velocity vary with time. It manifests itself in the form of large and small eddies, whirling about from one layer of fluid to another. A complete and mathematically rigorous theory of turbulence has not yet been developed. It is a basic principle that transverse velocity fluctuations follow a normal probability distribution. If turbulence is uniform in the direction of flow the energy present due to turbulence need not be taken into account in any energy equation. However, if the energy of turbulence varies from point to point, such as occurs in conduit expansion, beyond bends, etc. its omission, in any study of energy changes, results in an error. The energy of turbulence is dissipated into heat, due to viscosity, and cannot be recovered. The rate at which potential energy is transferred into turbulent energy is not, necessarily, equal to the rate of energy dissipation into heat at any one point. The turbulent energy can be diffused to other points by the action of the eddies and then dissipated into heat. Turbulence eddies are capable of causing the exchange of momentum, heat, matter, etc. from one layer of fluid to another. A knowledge of the diffusing power of turbulence is most important in studies of suspended material transportation. Experimental methods and equipment used to determine the various quantities involved in turbulence include: the hot-wire anemometer in wind tunnels and in water at low velocities; by photographing on moving picture film illuminated or colored suspended globules of carbon tetrachloride and benzine, having a sp.gr. equal to that of water, but imiscible with it; and by photographing on motion picture film a thin color stream injected into the flowing water through a fine needle. The cup type of current meter is sensitive to turbulence but it does not follow the velocity fluctuations exactly because of its inertia. In a conduit of expanding cross section, the additional losses of energy are directly attributable to the creation of turbulent energy beyond that ordinarily present in turbulent flow. The rate at which potential energy per unit length of circular conduit is converted into turbulent energy is:

$$Q\frac{h_f}{L} = 2\pi \int_0^r \tau \frac{dU}{dy}\, ydy \qquad \text{.....(21)}$$

in which, $\frac{h_f}{L}$ = loss of head per ft. of conduit; Q = total weight of water flowing in unit time; τ = unit shear stress at y distance from the center; r = pipe radius; and U = mean velocity parallel to pipe axis. In the problems of silt sedimentation the diffusion characteristics of turbulence are important. Two general problems to illustrate this are analyzed. The principles of turbulence may explain why surface velocity in open channel flow is less than maximum; the entrainment of air by a high surface velocity; and the relation of turbulence to the accuracy of the Allen salt-velocity method of measuring discharge. Instruments for measuring mean velocity and static pressure in a turbulent stream, with intensities of turbulence ordinarily present, are affected by turbulence but slightly, the anticipated errors due to turbulence being less than 1%. Turbulence should be thought of in terms of definite parameters instead of just as a qualitative descriptive term relating to a general condition of flow.—*H. E. Babbitt.*

ADMINISTRATION, PERSONNEL AND PUBLIC RELATIONS

Water Supply in War Time. Schedule of Reserved Occupations. ANON. Wtr. and Wtr. Eng. (Br.) **41:** 515 (Oct. '39). A list of reserved occupations has been issued showing the minimum reserved ages in several trades. The purpose is to ensure that workers required for maintenance of necessary production or essential services are not accepted for service in which their skill and experience will not be used. For example, some occupations, together with the min. age limit are: Water works administrative and superintendent, (25); reservoir keeper, foreman (25); service waste inspector, (30); meter fixer, (35); ferruleman, (25); bacteriologist, (25); and chemist, (21). Many other water works occupations are listed.—*H. E. Babbitt.*

Undergraduate Engineering Training in Public Health and Related Activities in Engineering Colleges of the U. S. ARTHUR P. MILLER. U. S. Pub. Hlth. Repts. **54:** 29 (Jan. 13, '39). In '36–'37 there were 25 colleges giving 26 courses in sanitary or public health engineering. Three of these lead to degrees specifically naming sanitary or public health engineering. The course subjects were quite varied. The number of graduates has increased materially since '16. Included are tables listing the schools, subject and percentage of time allotted, and number of graduates annually from 1889–1937.—*Ralph E. Noble.*

Life Insurance for City Employees. ANON. Public Management **21:** 252 (Aug. '39). Seventy-two employees of the city of Dusquesne, Pennsylvania, are now eligible to life insurance of $1,000 each, through a group policy recently acquired by that city. The policy involves a total of $72,000 and is of the contributory type, the workers paying a part of the premium and the remainder being assumed by the city.

Notification of Health Agencies re Water Works Emergencies. Connecticut Health Bul. **53:** No. 1 (Jan. '39). As an aftermath of experience with two major disasters affecting public water supplies to some degree, the Public Health Council at its meeting on Dec. 15, '38, enacted Reg. 128 of the State

Sanitary Code, which took effect on Jan. 3, '39. Copies of the regulation have been forwarded to all water works managements. While the cooperation of the water works officials with health agencies was excellent during the '36 and '38 disasters, the following regulation sets forth a definite required procedure to be carried out in the event of future emergencies: "Regulation 128. Whenever the treatment of a public water supply is interrupted or the source of the supply is damaged so as to impair the quality or the sufficiency of the supply, the person, firm or corporation, including municipal corporation, in charge of such supply, shall notify the state department of health and the local health officers of all cities, towns and boroughs where water from such systems is supplied. Such notification shall be made promptly either by telephone, messenger or whatever other means of rapid communication is available."—*P. H. E. A.*

Liability for Water-Borne Disease. EDITORIAL. Am. J. Pub. Health **29:** 378 (Apr. '39). It is well established in law that a purveyor of water for human consumption, whether a private water company or a municipal corp., will be liable for damages to consumers who contract diseases from the water as a direct result of negligence on the part of the purveyor. This legal doctrine should be familiar to all water works officials. Epidemic in '37 and '38 at Croydon, Eng., with 322 primary and 19 secondary cases of typhoid, with 43 deaths followed by court claims of over £100,000; in '28 an American court awarded $2000 to a minor and $1000 to his father for damages due to typhoid contracted from city water; Olean, N. Y. epidemic in '28 resulted in payment of claims amounting to more than 400,000. Prevention of disease in such instances not only necessary humanitarian duty, but is also an economic safeguard of vast significance.—*Martin E. Flentje.*

Warfare Gases and Municipal Sewage and Water Plants. ANON. Pub. Wks. **70:** 2: 21 (Feb. '39). Considerable study given to this subject in England; includes study of mustard gas and Lewisite which are both vesicants (attack skin) and lung irritants, Lewisite being also a systemic poison. Recommended that protection be provided plant operators both from high explosive bombs and gas from bombs. Protection as far as possible should be provided for machinery and equipment by sand bags and rubber hoods for pump bearings and other equipment requiring lubrication as both gases affect oils and greases. Contamination of reservoir waters with mustard or similar gases possible and protection or correction presents real problem. To prevent use of water from unsafe sources, complete treatment of the contaminated supply would be necessary and particular care given to production of blameless water from every aspect. Tests now available for detn. of presence of these gases in water are difficult, but hoped other methods under development will be suitable for general use. Treatment methods have been developed whereby all or practically all of contamination resulting from use of the more effective commonly used gases can be removed; these involve only slight modifications of standard practices.—*Martin E. Flentje.*

Selling Water to the Public. ANON. Eng. News-Rec. **122:** 501 (Apr. 13, '39). Newspaper advertising, outdoor advertising boards, welcome signs at

entrance to treatment plant and principal pumping stations, pamphlets, folders, calendars, blotters and other forms of printed material delivered by meter readers, and motion pictures are some of publicity avenues by which Indianapolis Water Co. endeavours to reach customers day by day. Relief maps of watershed area, model homes with water pipes and plumbing clearly shown and electrical map of underground distr. system are displayed around city and in office lobby from time to time. Publicity and advertising is but one step in public relations program. So-called public relations dept. was disbanded several yrs. ago. Company believes that every employee has public relations function in carrying on his every day duties and spreads that doctrine throughout organization. Satisfactory relations with customers depend basically on quality of product and service rendered.—*R. E. Thompson.*

Water Power Development in the Third Reich. OTTO KURZ. Deut. Wassertwirtschaft **34:** 113 (Mar. '39). The constant aim of the Third Reich to achieve a more beautiful Germany has had noteworthy results. It is demanded that future water power stations shall be not only more efficient, having fewer interruptions, and more simple, but also less conspicuous, harmonizing unobtrusively with the surroundings, and avoiding infringement of rights or amenities of other parties. Water power development in Germany in the past has usually taken the form of the side-canal, so that in summer when water is low it often happens that the main river bed runs nearly dry, and fish, water-fowl, and vegetation have perished, with devastating effect upon the landscape and upon ground water conditions. The side-canal is itself unsightly, and still more so the usual dams and piles dotting the river. Economy plainly demands that every water-power capable of being developed shall be set to work in order to save not only coal, but also the very heavy maintenance and renewal incidental to the coal-fired plant. It is stressed that for German conditions the side-canal is now out-dated and that the logical site for the power-house is in the stream bed. Further than this, recent developments have proved, not only the feasibility, but the great advisability of combining into a single submerged structure power-house, dam, overflow weir, and deep sluice gate; so that to the eye of the spectator the only change apparent in the river following the development is its embellishment by a new waterfall. Ministerialrat Arno Fischer's new plant on the Iller, conceived on these lines, has now been given a thorough trial with most satisfactory results. In comparison with what a plant of equal capacity and on the same site, but built in the usual style, would have cost to construct, the saving effected has been most impressive: on concrete work, 80%; on weight of steel, 60%; on time required for construction, 50%; and on total cost, 50%.—*Frank Hannan.*

Occupational Disease Hazard in the Public Utility Industry. J. J. WITTMER. Edison Elec. Inst. Bul. **7:** 9 (Jan. '39). In comparison with other industries, public utility industry relatively free from occupational disease hazards. Dermatitis and poison ivy mentioned, as well as cleansing materials. Personal sensitivity of certain individuals must be guarded against. Among volatile solvents carbon tetrachloride is extremely and dangerously toxic, when inhaled in concentrated form in space that is poorly ventilated, it will produce uncon-

sciousness with practically no premonitory symptoms, followed by almost immediate death. Silicosis not a problem in routine operation of industry. In av. street excavation work, laying mains, etc., author says it can safely be conceded that the drilling operations incidental to this work present no silicosis hazard.—*Martin E. Flentje.*

Personnel. PATRICK HEALY, JR. Public Management **21:** 111 (Apr. '39). State legislature of N. C. passed law on Apr. 3 providing for a state wide, centrally administered retirement system in which any city, town, or county might voluntarily participate for benefit of its employees. Requires approval of voters of community. Provides retirement and disability benefits financed on an actuarially sound basis by joint contributions of employees and employers. Retirement optional at age 60, compulsory at age 65. Legislation designed especially to meet problem of small city having too few employees to establish own actuarially sound system.—*Martin E. Flentje.*

Model of Filtration Plant Helps Educate Public. ANON. Am. City **54:** 5: 45 (May '39). Built for selling service, the St. Joseph (Mo.) Water Co. now has a miniature water purification plant in which as much as possible is made of glass so that the spectator may see the process of changing turbid raw water into sparkling drinking water. Description of this tiny plant is in sufficient detail that it constitutes directions for building.—*Arthur P. Miller.*

On Attaining Success in Water Works Operation. REEVES NEWSOM. W. W. & Sew. **86:** 159 (May '39). Any young man can become a successful operator if he handles public and personnel relations well, studies trade publications, attends water works meetings, visits other plants, and enters into community activities.—*H. E. Hudson, Jr.*

Water Supply for "Domestic Purposes." ANON. Wtr. and Wtr. Eng. (Br.) **41:** 248 (May '39). The Court of Appeal found that the use of water was "domestic" under the law, whether or not it was mixed with medicine or whiskey, as is also water used for washing anything, including water for washing of horses and carriages for private use. A large number of previous court decisions are cited in which these legal principles are upheld.—*H. E. Babbitt.*

Contract for Engineering Service. ANON. W. W. Inf. Exch., Canadian Sect. A. W. W. A. **3:** D: 1: 1 (May '39). Text of judgement in case of Fleming v. Town of Tillsonburg, Ont., is given. Engineer, since deceased, had been instructed to investigate washout of dam used for storing water for fire protection and subsequently to prepare plans and specifications and call for tenders. Later he was instructed to withold calling tenders and another engineer was asked by corporation to collaborate. Subsequently, fresh plans and specifications were prepared by Fleming, incorporating suggestions of collaborating engineer. Fleming's estate sued corporation for payment and was awarded judgement for $1,578.72 and costs. Charge for preparing plans and specifications was reduced from 4 to 3% by court.—*R. E. Thompson.*

Bonded Debt of 289 Cities as of Jan. 1, '39. ROSINA MOHAUPT. Nat. Munic. Rev. **28:** 438 (Jun. '39). Decrease in bonded debt of such cities over 30,000 for which figures for '34 and '39 are available is attributed to stationary population, federal aid and conceivably to "anxiety over the high cost of government, reflected in a reduction in costly municipal public improvement." For 208 cities reporting both years (New York excluded) the gross bonded debt fell from $5,274,000,000 to $5,244,000,000 or from $142.05 to $141.25 per capita. Excluding municipal utilities the corresponding figures were more notable: $3,534,000,000 to $3,423,000,000 or $95.19 to $92.20. Utilities include water, light and power, street railways, port and harbor facilities, docks, and sewerage. Prior to '39 debts of "self-supporting utilities" only were deducted from gross to determine net indebtedness. Difficulties and uncertainties in deciding whether utilities were self-supporting led to abandonment of the attempt and the use of total utility debt instead. In Detroit, the water works meets all debt and operating charges, supplies water free for all public buildings and municipal services, operates a large office building for other city departments but pays no taxes. The city street railway system pays taxes on real and personal property. The city-owned electric light plant lights all public buildings, without charge, except the water works building. In another city, the water utility is considered self supporting, although the city pays all its debt service, performs all overhead functions and collects no taxes from the water works; but the city pays about $100 hydrant rental and for all water used for city purposes. In many cities utility rates are political footballs; increases, especially for water, are avoided and deficits are made up from other revenues. In addition, a utility is classed self supporting one year and not so the next. By deducting all utility debt from gross to find net debt the yearly debt figures will become more truly comparable. ★[Space has been given to the inconsistencies just noted in order to call attention to the need of reform in city financing, which is notably true of many water utilities. In some cities, consumers pay not only for the entire cost of the water service but also for a considerable part of other city services. Every municipal tub should stand on its own bottom.]★—*M. N. Baker.* (*See also abstract* J. A. W. W. A. **30:** 1574 (Sep. '38)).

Conditions under Which Water Service Should Be Extended. ANON. Pub. Util. Fort. **24:** 59 (Jul. 6, '39). Penn. commission held water company not required to extend domestic water service to real estate subdivision, if a deficit showed on additional operating expenses, and return on additional capital invested was 1.81%; company was willing to extend service if 3.41% could be obtained. Commission ordered company to extend service if sufficient domestic and fire protection service were received to yield 2.68%; difference between this return and company's demand one of minor degree only, particularly as community showed prospective growth. Each extension must be reasonable, for prospective consumers and the utility, considering circumstances of all concerned. Complaint that supervisors erred in not entering into contract with water company dismissed, commission disclaiming authority to adjudicate differences between supervisors and their constituents.—*Samuel A. Evans.*

Indiana Appelate Court Ruling. ANON. Pub. Util. Fort. **24:** 62 (Jul. 6, '39). Held Commission can establish rules and regulations for government of utilities, for prosecution of business, but cannot relieve them from liability under the law of negligence by any rule the commission may adopt.—*Samuel A. Evans.*

Water Bills Liens Against Property. ANON. Public Management **21:** 218 (Jul. '39). Michigan's legislature recently passed a bill permitting liens against property for water bills, a practice which many Michigan cities have followed for decades. A recent court decision, however, had held the lien system invalid because of the absence of a specific state enabling act. . . . . The Indiana Board of Accounts recently ordered officials of the city water works of Alexander, Indiana, to serve notices upon delinquent consumers, and to turn off the water unless payments are immediately forthcoming.

Public Opinion by Post Card. STUART M. WEAVER. Public Management **21:** 138 (May '39). Montclair, N. J., a suburban residential city of 45,000, polled by post card (using water dep't. mailing lists) regarding curtailment of municipal expenses. Citizens in 25% return did not approve reduction of health and safety services of city but were willing to accept reduction in services of "convenience" nature, as library, tennis courts and frequent refuse collections. Some of recommendations put into effect.—*Martin E. Flentje.*

How to Prepare Office Manuals. JOHN F. PIERCE. Pub. Wks. **70:** 9: 23 (Sept. '39). Because of desirability of having standard office practices and to prevent any regression from adopted standards, written instructions in the form of office manuals are considered necessary. Suggestions for preparing such manuals, based on experiences in TVA organization, are given. Manual to be of greatest value must give information regarding policies, organization and procedures of the group it is to serve. Text generally in narrative form and in third person; simple future tense used frequently to describe a procedure in tone of an indirect command; term "should" expresses obligation, "must" used sparingly so as not to lose importance, indicates a mandatory ruling. Manual to be useful must be kept up-to-date.—*Martin E. Flentje.*

Water and Sewerage System Safety. CARL A. HECHMER. Md.-Del. W. and Sew. Ass'n. **13:** 27 ('39). Men employed in notifying water consumers that water supply to house must be shut off during main repairs should be qualified to aid housewife in operation of furnace or hot water heater during shut-off, and should caution her not to draw water out of hot water heater but to relieve steam pressure that may develop by occasionally opening upstairs hot water faucet. Incident described in which restaurant counterman burned by hot water and steam from coffee urn caused by either air from main or quick generation of steam when water turned back on. After main repair, charging main too quickly and improper release of air are potential causes of many accidents. Providing goggles for man out on ditch who must cut pipe or paving is good safety practice. Safety gates used by Washington Suburban Sanitary District around open ditches described. Lanterns should not be

filled while lighted, although this is often done. Automobile safety meetings have reduced accidents.—*Martin E. Flentje.*

Partial Fire Insurance Survey Shows Favorable Loss Ratio. ANON. Minnesota Municipalities **24:** 373 (Oct. '39). Partial returns are tabulated of survey showing ratio of fire loss claims received to premiums paid on municipal buildings and contents in 39 Minn. municipalities, pop. 2,500 to 101,000, av. 8,500. Total 481 years experience. Of $357,785 premiums 22.65% received back in loss payments. Stated that similar surveys elsewhere have shown even lower percentages,—15 to 20%. These data compare with average loss ratio of stock insurance companies of about 50%, and previous Minn. tabulation of all types of risks of 49%, to show municipal buildings to constitute a preferred fire insurance risk. ★[This abstractor has long contended that water works buildings in particular were better than average fire risks, and that some means should be found of securing insurance at lower cost]★.—*P. S. Wilson.* (*See also abstract* J. A. W. W. A. **31:** 389 (Feb. '39)).

Relative Value of Utility Systems. C. F. LAMBERT. Eng. News-Rec. **122:** 582 (Apr. 27, '39). Records of values of 5 utilities (waterworks, electric light plants, street railways, natural and artificial gas plants) originally published in Eng. News-Rec., May 7, '25 and brought up to date annually since '31, are extended to Jan. 1, '39. About twice as many plants as originally are now used in making computations. Yearly values are tabulated as percentages of '13 and '26 values and monthly values during '38 as percentages of '13 value. Latter show definite decrease during latter half of yr. Values, using '13 as base, are also shown graphically. Data for waterworks are based on 25 systems, 68 items used including land, buildings, equipment (boilers, pumps, etc.), distribution (pipe, valves, hydrants, meters, standpipes, reservoirs, filter plants, etc.), and miscellaneous. Taking '26 value as 100, yearly value (waterworks) dropped as low as 81.0 in '32; in '36, '37 and '38 it was 94.5, 104.3 and 102.1, respectively. On basis of '13 value as 100, '36, '37 and '38 values were 178.0, 195.5 and 192.6, respectively.—*R. E. Thompson.*

Contract for Wholesale Service Can Be Terminated Without Commission Authority. Benwood-McMechen Water Co. v. City of Wheeling, 4 S.E. (2d) 300. Pub. Util. Fort. **24:** 631 (Nov. 9, '39). West Virginia Supreme Court of Appeals rescinds order of Utilities Commission. If city agrees to sell water to private water company for an indefinite period, service may be terminated after reasonable notice by either party. Dispute over rates, city desired to discontinue service. Public not parties to the contract. City by charter, water company by franchise, each obligated to furnish service in particular area, one could not encroach upon the other, therefore neither could call upon the other to perform that which it was obligated to provide.—*Samuel A. Evans.*

No Service Denial to Enforce Payment for Fire Protection. Re Anthracite Water Co. (Application Docket No. 53771). Pub. Util. Fort. **24:** 652 (Nov. 9, '39). A water company cannot discontinue fire protection service to a munic-

ipality upon the latter's failure to pay for same, Pennsylvania Commission ruled. Legislative reason clear, ordinarily utility not required to continue service when the money involved is less than cost of attorney's fees and litigation to collect same, as in case of residential patron. Foregoing reasoning does not apply to municipality, since generally bill would be more than cost of collecting same. Another reason: fire protection is a municipal government benefit, failure to provide is a failure of government itself. In an emergency police power will permit government to commandeer necessary protection for citizens from fire, flood or other public disaster.—*Samuel A. Evans.*

Pennsylvania Public Utility Commission. Re Pittsburgh and Shawmut Railroad Company. Commission Ruling. Pub. Util. Fort. (Nov. 9, '39). P. U. R. **30:** 191. Railroad applied for abandonment of a branch line, which crossed a stream on a trestle. Witness for Water and Power Resources Board testified that, trestle should be removed, and a clear stream bed 32 feet in width, be made part of order. Commission stated, "nothing in the Public Utility law gives us jurisdiction over streams and their flow." Matter of such jurisdiction with the Water and Power Resources Board.—*Samuel A. Evans.*

No Formula for Choosing Good Corporation Directors. ALBERT W. ATWOOD. Pub. Util. Fort. **24:** 595 (Nov. 9, '39). In any large business, especially the utility business, from standpoint of internal management and public relations composition of board of directors of prime importance. Excess of over 100,000 corporate directors in this country, little known about methods of choosing. Incorporation laws, govern qualifications of directors, directors have great powers, state legal qualifications few and unimportant. Utility directors have been favorably and adversely criticized. Numerous restrictive measures pertaining to interlocking directorates and upon bankers as corporate directors. Recent Walker report, by member Federal Radio Commission claims president of Am. Tel. and Telegraph selects directors with executive committee and existing directors. Legally, directors should select president rather than the reverse, however directors not selected by stockholders. Impractical for small owners of stock widely scattered to choose directors. In utility interest where public relations are important, conflicting trends exist, some large organizations have nationally known figures, some substantial local citizens. Question ultimately should directors be "inside" men qualified by experience or "outside" men who will act against managerial abuse. What to do about it? William O. Douglas, former Chairman Securities Exchange Commission now associate justice United States Supreme Court, states boards of directors made up of "outside" directors who theoretically guide and control and "inside" who are the management. "Outside" directors have superficial knowledge hence management governs. So called strong man, representing great bank or insurance company, naturally dominates others. If man is director as an avocation, deriving no income, he follows management who seem to speak with authority. Douglas' remedy, he quotes Joseph P. Kennedy, Ambassador to Great Britain, "directors responsibility greatly exceeds honorarium." Douglas further states, corporations endeavor to obtain something for nothing, then sometimes directors attempt to collect

invisible rewards; inside information used for market tips, their banking connections, and otherwise. Countries outside United States directors receive salaries, developing a professional director, frequently of high capacity. Disadvantages, sometimes in Great Britain directorships sold, similar to medical practice, price five times annual income. Probable solution, directors large stockholders. No definite formula, same old story, the difficulty of finding the right man.—*Samuel A. Evans.*

Report of the French Commission to the Congress of Liége, July 3–6, 1939. Regional Grouping for Public Supply of Potable Water. VIGNEROT. Tech. Sanit. Munic. (Fr.) **34:** 127 (Aug. '39). *Ch. I. Considerations on Collective Grouping in View of the Common Public Service. Generalization of Intercommunal Syndicates.* Communal services, such as distribution of electricity, gas or water, or sewage treatment, can be administered directly by the governing body concerned or through concessionaires. Formation of communities into groups for some of these functions makes possible better legal, sanitary and technical control than each could have alone. *Ch. II. Particular Characteristics of Collective Grouping for Water Supply. Ibid.* **34:** 129 (Aug. '39). VIGNEROT: In France, the State intervenes in syndicates of communities organized for water supply only to exercise technical and sanitary control. Plans for new systems must have official approval before construction begins. Some public funds are available for preliminary investigations of sources of supply. Advantages of grouping of communities for water supply include economy of construction, possibility of serving isolated houses and farms along pipe lines, rational linking of distribution systems, possibility of lowered rates due to increased service connections, assurance of proper maintenance of system by qualified personnel, collection of bills and economical management facilitated by its own prestige, rational utilization of water resources of a region, increased ease of obtaining working capital, freeing of community authorities from responsibility for maintenance of system, ease of control by public health authorities and simplicity and economy in allocating public grants. *Ch. III. Study of Various Cases in Grouping Communities for Water Distribution. Ibid.* **34:** 130 (Aug. '39). PINSET AND SALMON: (1) Grouping is indispensable for creation of supply where single well or well site is available for several communities, single treatment plant will suffice, district is densely populated or irrigation needs can be met by cooperation. (2) Grouping is not necessary but presents indisputable technical or economical advantages where group can share important well or well-site, group project will effect economies in construction, communities lie adjacent to large cities and can thus obtain water supply, community lies along important pipe line which can be tapped, cooperating communities can obtain technical and sanitary services not otherwise obtainable, in both construction and maintenance. (3) Grouping is not advisable where communities lie too far apart, population is too sparse or greater altitude of some of the communities places increased pumping costs on lower-lying villages. (4) Chief obstacle to formation of syndicates is unwillingness of local authorities to delegate power to an inter-community organization, particularly since community at same time engages to underwrite fixed portion of costs over long period of years. This

objection is partly overcome when work is entrusted to private concessionaire. *Ch. IV. Evolution of Water Distribution and Regional Groups. Ibid.* **34**: 134 (Aug. '39) DONZET: Modern water distribution systems in France began in middle of last century though most pre-war systems were incomplete or provided too little water for present needs. Much of pop. was formerly served by public hydrants; trend now is toward separate house connection for each subscriber. Pipe sizes are now chosen with due regard for fire protection. Of present community groups for water supply, only one was begun before 1914; between 1920–1930 only 4 were organized; at present 52 groups serve nearly 400 communities with pop. of around 300,000. Expansion is due to services rendered by Minister of Agriculture, studies initiated by engineers, State or departmental subsidies, influence of general economic conditions, recognition in rural regions of advantages of water supplies, extension of rural electrification schemes to include water projects, pumping economies possible after electrification of communities, technical advances in water supplies, and extension by concessionaires of existing supplies to serve adjacent communities. Appended are 2 large tables giving (1) for existing group installations, data on no. of communities and pop. served, purification procedure if any, type of system, cap. of reservoirs, characteristics of distribution system, type of management and supervision, water rates, mode of financing and reasons for formation of group; (2) for proposed group installations, no. of communities and pop. served, status of project, subsidies granted, proposed cost, type of installation, purification procedure if any, proposed management and supervision, and reasons for formation of group. *Ch. V. General Guides for Study, Realization and Exploitation of Regional Groups for Water Distribution.* DONZET: Ends to be attained are: distribution of water of best quality to all who can be economically served, division of program into sections to be progressively realized if necessary as units of whole, choice of best solution to assure economical exploitation without sacrificing security and continuity of project. This necessitates careful preliminary studies of all phases of project, e.g., availability of supply, type of construction, cost and financing, nature of communities to be served. Studies made over last 10 years show av. daily per capita consumption at 60 liters, and 120 liters should insure good service. No. of wells should be kept small and pumping dispensed with if possible. Construction, maintenance and management should be simple, flexible and economical. Pipe sizes should be large enough for future needs and planned with fire protection in mind, though use of reservoirs allows reduction in pipe line diameters. Elevated reservoirs should be carefully designed for utility and conformity to surroundings. Secondary reservoirs may be needed for each individual community. Cap. should be 1½ to 2 times daily consumption. Valves should be imbedded in masonry to facilitate locating and handling. Enough hydrants should be available for fire protection but wide use of public hydrants by householders decreases revenue from water sales. Minister of Agriculture urges extension of water service to isolated locations along pipe lines to serve largest possible pop. Financing must be carefully planned in advance. Author recommends flat rate for fixed min. of water with excess charged at reduced rate.—*Selma Gottlieb.*

Stabilized Method of Forecasting Population. BERNARD D. KARPINOS. Pub. Hlth. Repts. **54:** 1807 (Oct. 6, '39). Broadly viewed, there are two types of population (pop.) estimates from the standpoint of time, i.e., those which reconstruct the past or forecast the future. The first is an evaluation of the actual pop., while the second tries to determine what could be expected if certain pop. factors operating in the past should continue in future. Actual pop. estimates may be, (1) *precensal*, i.e., for yrs. prior to reliable census data and determined by extrapolation; (2) *intercensal*, between two census dates, by arithmetic or geometric interpolation; and (3) *postcensal*, after a census date, but, (a) in the absence of birth and death records, or (b) where such records are available. In 3 (b), when possible to obtain fair estimates of net migration, it is known as the "migration and natural increase method." It is less valuable for application to city or state data because of indeterminate internal migration factors. In forecasting future pops. one may fit a curve to data for the past and project it into the future, but, thus the determining factors are not analyzed and the end result is the only guide. Another method, which does take into account essential factors, is described by the author and is known as the, "stabilized method of forecasting populations." It consists of starting with a pop. of a certain point in time, usually the latest census with its age, sex, race, and nativity distribution. No law of pop. growth is sought in this case. By assigning to the selected pop. different birth and death rates and different net migrations, variously combined, there are gradually built up different pop. forecasts. The assigned birth and death rates, as well as the net migrations, are taken within reasonable limits of credibility determined by the prevalent trends. The building up of the pops. is done essentially on the basis of survival factors. This method is adaptable mainly for long-time forecasts. It is simple and flexible, eliminates the laborious procedure involved by the previous method, and it utilizes the stabilized age compositions which, however, are hypothetical structures. The latter are derived on the assumption that pops. grow freely under the influence of actual or postulated fertility and mortality, without disturbance by emigration or immigration. "Left alone" they will ultimately conform to fixed age structures and grow at uniform rates. Such ultimate age compositions are termed, "stabilized age compositions" and the rates called, "true" rates of natural increase. Tables showing the application of the method to types of pops. differing with respect to their true rates of natural increase and prevailing age compositions show that within 2 generations, about 60 yrs., such pops. will practically attain the theoretically computed stabilized age compositions, assuming fertility and mortality remain unchanged. Of course, constancy of either for such a period is unexpected. Yet the fact that such an adjustment could practically be accomplished within 60 yrs. or less, becomes the underlying principle of the method. Concrete examples are cited. Included are 8 tables, 6 figs., 17 refs.—*Ralph E. Noble.*

STERILIZATION

Differentiating Between Chlorine and Chloramine Residuals. A. E. GRIFFIN. Md.-Del. W. and Sew. Ass'n. **13:** 65 ('39). Methods for detn. chlor-

amines involving bleaching of methyl red and measurement of ammonium ions following acidification have not been found generally useful. Author proposes use of sodium nitrite test depending upon ability of free chlorine to oxidize nitrites rapidly and inability of chloramine to do so. Cites tests confirming assumptions. In making test, residual is detn'd, in usual way, then 2 tubes filled to mark and sodium nitrite solution equivalent in p.p.m. to 1½ times chlorine found is added to 1, followed by addition o'tolidine to both and standing in dark for full color development; then residual detn'd. Difference in residuals shows chloramine present.—*Martin E. Flentje.*

Color Tests for Chlorine, Ozone and Hypochlorites with Methane Base. A. T. MASTERMAN. Analyst **64:** 492 ('39). The compd. 4,4'-tetramethyldiaminodiphenylmethane is called methane base. It dissolves readily in EtOH, CCl_4 or AcMe. Addition of O_3 to soln. of methane base gives series of colors: violet, amethyst, rose and ruby red. Addition of Cl_2 gives colors: blue, grass-green, olive-green, orange, yellow and finally complete bleaching. Hypochlorites at first give same sequence of colors as Cl_2. They can be made to give O_3 sequence by concg. methane base in EtOH. In aq. solns., electrolytically prepd. hypochlorites give Cl_2 colors and then O_3 colors. Chemically prepd. hypochlorites do not give O_3 colors except after standing for some time but addition of NaCl assists in formation of color. Electrolytically prepd. hypochlorites decolorize fluorescein more rapidly than do those prepd. chemically. It is possible that electrolytic products contain O_3.—*C. A.*

Chlorine plus Ammonia—How, Why, When, Where. A. E. GRIFFIN. Penna. W. W. Oper. Assoc. **10:** 88 ('38). When the addn. of NH_3 to the water to be treated is followed by the addn. of Cl_2, chloramines are formed. The proportion of NH_3 to Cl_2 in water of pH 8.4 for monochloramine is 1:4, at pH 4.8 for dichloramine it is 1:8. The av. in many plants is 1:3. For a rapid sterilization the Cl_2 is added first. Cl_2 to the extent of 25 p.p.m. may be added to a by-passed portion from the main, then NH_3, followed by reuniting the 2 streams. The initial dose of Cl_2 sufficient to give a certain residual within a few min. is approx. the same with or without NH_3. With NH_3 the water is able to retain the residual Cl_2 for a much longer time. A final 1.0 p.p.m. Cl_2 (with NH_3) is a very effective sterilizing agent, and controls the organisms of the coli-aerogenes group as well as prevents growth of slime deposits and iron bacteria. Chloramine, or better cuprichloramines, are very effective in algae prevention in settling basins. The Cl_2 content is detd. by the *o*-tolidine test at 70–100°F.—*C. A.*

New Automatic Chlorinator Station. GORDON L. O'BRIEN. W. W. and Sew. **86:** 271 (Jul. '39). Station installed to chlorinate outflow from balancing reservoir at Baltimore. Chlorine feed automatically proportioned to water flow by use of Pitot meters. Plant includes differential converters actuating chlorinators, automatic vacuum solution feed chlorinators, recording scales, careful heating and temperature control for cylinder room and tray water, ventilating system, and numerous safety devices.—*H. E. Hudson, Jr.*

A Critical Study of the Thiosulfate Titration of Chlorine. F. J. HALLINAN AND W. R. THOMPSON. J. Am. Chem. Soc. **61:** 265 (Feb. '39). This study, made in laboratories of N. Y. State Dept. of Health, was suggested by statement in appendix to '36 "Standard Methods," of need for work on iodine-thiosulfate titration in pH range of natural waters, to detn. available chlorine under actual conditions of use. Effect of iodide concn. and interfering substances, esp. oxidized manganese, was also studied. Max. titer of Cl solns. was obtd. at approx. pH 1.3 with over 1 gram of KI per liter of sample. With pH between 4.6 and 7.7, and KI concn. between 0.05 and 100 grams per liter, results under exptl. conditions described may be approximated by:

$$\log \frac{1 - R}{R - 0.125} = 0.02806\,[20\ \mathrm{pH} - 14 \log \{\mathrm{KI}\}] - 4.260$$

and:

$$\log t = 0.6\ \mathrm{pH} - \log \{\mathrm{KI}\} - 2.383,$$

where R is ratio of observed to max. titer, {KI} is grams of KI per liter of sample, t is time required to produce sufficient I by introduction of 0.5 p.p.m. of Mn (in a sol prepd. from $MnSO_4$ and $KMnO_4$) to color an α-naphthoflavone indicator as much as would 0.05 p.p.m. of Cl under the standard conditions of pH and KI concn. ★[Units of time not given by authors]★. These equations have been used to develop a nomogram which may serve as guide to Cl titration with thiosulfate in presence of MnO_2. By taking advantage of the moderate diff. in slopes of the R and t nomogram lines, it should be possible to choose conditions where SO_4 formation and Mn interference are negligible. In exptl. titrations, solns. were usually buffered with ammonium acetate. Thiosulfate was 0.005 M and main study was made on 40 ml. samples of solns. contg. 7 to 12 p.p.m. of Cl. Extensive study of possible interference by nitrite and ferric oxide was not made, but indications are that conditions sufficient to repress MnO_2 interference also repress interference by nitrite and ferric oxide. Unfortunately, there appears to be no combination of pH and KI concn. available which eliminates risk of liberation of I by bactericidally ineffective Cl derivatives present, e.g., organic chloroamines or Cl adsorption products. If low titration is obtd. thru improper adjustment of pH and {KI}, correct titer cannot be obtd. by further additions of acid, KI or both.—*Selma Gottlieb.*

Industrial Aspects of Chlorine. PAUL S. BRALLIER. Can. Chem. and Process Industries **23:** 111 (Mar '39). The '21 production of chlorine in U. S. was 67,000 tons, which exceeded demand, while in '38 market absorbed 420,000 tons. Industry is equipped to produce approx. 595,000 tons annually. Electrolytic processes accounted for production of all but 5,000 tons of output in '38. Largest proportion of electrolytic chlorine (340,000 tons) is made from sodium chloride brine, principally in diaphragm cells. Remainder of production is divided as follows: electrolysis of fused salt, 65,000 tons; electrolysis of potassium chloride, 10,000 tons; and by action of concentrated nitric acid on salt, 5,000 tons. Careful attention to reconditioning of valves, etc., has kept

transportation of chlorine free from serious accidents. Following failure of tank-car, due to crystallization of steel, specifications were modified to prevent re-occurrence, and tank-cars have even been involved in wrecks without loss of chlorine. Oxidizing power of ordinary bleaching powder, $CaOCl_2$, is generally expressed in terms of available chlorine (35%), although it is really the oxygen content that does the work. Thus newer, high-test hypochlorites, $Ca(OCl)_2$, have oxidizing power which may be expressed as 65–70% available chlorine, although actual chlorine content is only 36–40%.—*R. E. Thompson.*

Cost Basis for Using Chlorine as a Liquid or as Hypochlorites. H. B. Foote: W. W. Eng. **92:** 255 (Mar. 1, '39). "Break-even point" or point at which 2 alternatives become equally economical may be used to detn. proper form of chlorine to use. Formula suggested: $X = OPN+T+(I+D+M)C$; in which X = annual cost, C, first cost; I, annual interest on investment; D, annual depreciation and obsolescence; M, annual upkeep and repairs; O, cost of disinfectant; P, lbs. disinfectant used per day; N, days used; and T, interest on deposit on Cl tank. By plotting curve, with total annual cost and av. daily dose in lbs. for both chemicals as coordinates break-even point and most economical method can readily be detnd.—*Martin E. Flentje.*

Bathing the Green Goddess. Purification of Chlorine. Walker Penfield and R. E. Cushing. Ind. Eng. Chem. **31:** 377 (Apr. '39). Field investigations of "impure chlorine" complaints show most chlorinator plugging due to maintenance errors or negligence. However necessity is recognized of removing from chlorine small amts. (less than 0.02%) of chlorinated hydrocarbons causing "taffy" residue in chlorinating equipment. Authors describe special purification step introduced into Cl mfg. process by Pennsylvania Salt Mfg. Co. Cl is passed through special fractionating column which removes impurities as a solution with little or no loss of Cl. Improvement in buyer-seller relations is considered to justify additional processing cost.—*Selma Gottlieb.*

Refinement in the Preparation of Permanent Chlorine Standards. R. D. Scott. Ann. Rept. Ohio Conf. Water Purif. **18:** 58 ('38). Chromate-dichromate standards previously described (Standard Methods, 8th ed. p. 232) have been used during past 3 yrs. with satisfactory results. They are superior to dichromate-copper sulfate standards in that they exactly simulate the chlorine-*o*-tolidin color and may be compared in any depth of liquid. Buffer mixture recommended, however; deposits white sediment after a time. This may be avoided by substituting following buffer solution: (I) Boric acid, 12.4 grams made up to 1 liter with dist. water. (II) Borax, $Na_2B_4O_7 \cdot 10H_2O$, 3.8 grams made up to 1 liter with dist. water. (III) Sufficient vol. of II, usually 80–90 cc., to 1 liter of I to give pH 6.5. Standards prepared by using III to make required volumes of chromate and dichromate solutions up to 100 cc. are permanent for at least 1 yr. when kept in dark.—*R. E. Thompson.*

The Action of Chlorine on Bacteria in Water in the Presence of Various Organic Substances. Gunnar Nordgren, Per Funkquist and Sven-Gunnar

ANDRÉN. Acta Path. Microbiol. Scand. **16:** 1 ('39). In pure water a combination of Cl and NH_3 was more efficient than Cl alone. In the presence of org. matter Cl alone was more effective. Different types of org. matter influence the action of Cl differently. Mutation of coli may occur in the presence of Cl.—*C. A.*

The Germicidal Action of the Halogens on Anthrax Spores. E. HAILER AND VON BOCKELBERG. Arch. f. Hyg. u. Bakt. **122:** 20 ('39). In estimating the germicidal efficiency of the halogens the authors chose Krönig and Paul's garnet method. Anthrax spores were dried on garnets. The garnets were immersed for a given time in the solution of disinfectant to be tested; they were transferred for ten minutes to a sodium sulfite solution, which neutralized the remaining disinfectant; they were then suspended in a known quantity of water, and submitted to mechanical shaking for 5 min. to detach the spores. Finally the washings were counted by the plate method. The ordinary suspension method of testing disinfectants is regarded as unsuitable for highly active substances like the halogens. Either a weak suspension of spores must be used, in which case there will be so few resistant spores that the germicidal activity of the substance will appear too high; or a strong suspension must be used, in which case some of the disinfectant will be bound by the organic material which is always present in bacterial suspensions, and the germicidal activity will appear too low. A comparison of isomolecular solutions showed that iodine was more active than chlorine, and chlorine than bromine. The difference in activity, however, between chlorine and bromine was not very great and the order was not always constant. The temperature coefficient of the halogens was very low. Little difference was found between their activity at temperatures of 4°C., 12°C. and 20°C., though killing was rather more rapid at the higher than the lower temperatures. The concentration coefficient was also found to be low. A tenfold increase in concentration of a chlorine solution from 0.005% to 0.05% reduced the time taken to sterilize the suspension only threefold, that is from 3 hrs. to 1 hr. Alcoholic solutions of iodine were found to be active provided some water was present, but in pure organic solvents like carbon tetrachloride the germicidal action of the halogens was considerably diminished. On the whole the authors think that the germicidal activity of chlorine has been rather over-estimated by previous workers.—*B. H.*

The Lethal Action of Short Ultra-Violet Rays on Several Common Pathogenic Bacteria. D. G. SHARP. J. Bact. **37:** 447 ('39). Several species of pathogenic bacteria were exposed, as plate cultures, to ultraviolet radiation with the predominant wave length 2537 Å. A mixture of spores and vegetative cells of *B. anthracis* was not more than twice as resistant as *Escherichia coli*. 90% of the non-sporing *Shigella paradysenteriae* were killed by radiation with an energy of 168 ergs per sq. mm.; the lethal value for the most resistant non-sporing species was 337 ergs per sq. mm. for *Coryne-bacterium diphtheriae*. Results shown in graphs and compared with those of other observers. —*W. P. R.*

Super- and De-chlorination Practices in Water Treatment. A. I. Andrews. Off. Bull. N. D. Wtr. and Sew. Wks. Conf. **6**: 7: 9 (Jan. '39). Probably the most widely used methods for removing and preventing objectionable tastes and odors in domestic water supplies, are the ammonia chlorine process, activated carbon, or combination of both. However, these processes are not cure-alls in every taste and odor problem. Another method is the use of super- and de-chlorination. Chlorine added in relative high concentrations, (some cities as much as 12 p.p.m.) then a sufficient contact period before de-chlorination to allow the chlorine to oxidize completely the taste producing materials, have proven successful in many cities. The most commonly used materials for de-chlorinating are sulfur dioxide, sodium bisulfite, sodium thiosulfate and activated carbon. Water containing phenols, iron, crenothrix growth, hydrogen sulfide, algae, and highly polluted with industrial waste have all been successfully treated with this method.—*P. H. E. A.*

M. P. Otto. 1870–1938. L. Descroix. L'Eau (Fr.) **32**: 45 (Apr. '39). Obituary sketch. M. Otto installed first ozone water purification system in 1905 at Nice, his native city.—*Selma Gottlieb.*

Combating Iron Bacteria and Fungi by the Cuma-System. C. J. Oosterholt and J. Rodenburg. Water (Netherlands) **23**: 191 (Nov. 3, '39). Difficulties with slimes caused by iron bacteria and fungi at Eindhoven resulted in grey, dirty water at dead ends, decrease in size of narrow distribution lines, clogging of Venturi and Waltmann meters, floating of slime in clear wells. Troubles began after iron removal was practiced, probably because the slow after filters retain spores. Chlorination had no effect, but caused taste. The Cuma (cum argento) system consists of weak solutions of silver produced by electricity. The anodic silver chloride solution is very active and oligodynamic (germ killing) so that after 14 days all slimes disappeared with a dose of 30 grams silver per 264,000 gal. water. A small apparatus, consisting of a small wire seive to retain bacteria and spores and through which a constant stream of water flows, connected with a simple manometer, regulates the dosage. Interruption of treatment to apply copper sulfate resulted in slime, which again disappeared in two weeks after resuming silver treatment.—*Willem Rudolfs.*